BROWN BRIDGE
National Covered Bridges Recording Project
Spanning Cold River, Upper Cold River Road
Shrewsbury
Rutland County
Vermont

HAER VT-28
VT-28

PHOTOGRAPHS

WRITTEN HISTORICAL AND DESCRIPTIVE DATA

REDUCED COPIES OF MEASURED DRAWINGS

FIELD RECORDS

HISTORIC AMERICAN ENGINEERING RECORD
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HISTORIC AMERICAN ENGINEERING RECORD

BROWN BRIDGE

HAER No. VT-28

LOCATION:	Spanning Cold River, Upper Cold River Road, Shrewsbury, Rutland County, Vermont. UTM: 18/668056/4825804
DATE OF CONSTRUCTION:	1880
STRUCTURAL TYPE:	Town lattice truss
DESIGNER/ BUILDER:	Nichols M. Powers
PRESENT OWNER:	Town of Shrewsbury
PREVIOUS AND PRESENT USE:	Public road bridge since its construction
SIGNIFICANCE:	Brown Bridge is one of the best-designed examples of the Town lattice truss, a widely used style of timber bridge framing. It is also notable as the last bridge by Nichols M. Powers of Clarendon, one of Vermont's best-known bridge builders.
AUTHORS:	Joseph D. Conwill, Historian, July 2002. Dylan Lamar, HAER Engineering Technician, and Benjamin Schafer, Ph.D., Assistant Professor of Civil Engineering, Johns Hopkins University, Summer 2002.
PROJECT INFORMATION:	The National Covered Bridges Recording Project is part of the Historic American Engineering (HAER), a long-range program to document historically significant engineering and industrial works in the United States. HAER is part of the Historic American Buildings Survey/Historic American Engineering Record, a division of the National Park Service, U.S. Department of the Interior. The Federal Highway Administration funded the project.

CHRONOLOGY

1761	Shrewsbury is chartered
1780s	First major wave of settlement
1784	Birth of Ithiel Town
1817	Birth of Nichols M. Powers
1820	Ithiel Town patents the Town lattice truss
1835	Ithiel Town's revised patent
1837	Nichols M. Powers' first bridge at Pittsford Mills
1844	Death of Ithiel Town
1880	Nichols M. Powers builds Brown Bridge
1897	Death of Nichols M. Powers

SHREWSBURY: THE TOWN AND ITS RIVERS

Shrewsbury, Vermont is mostly forest today. The northwest half of town, nearest Rutland, is residential; the southeast half still has remnant pasture from the older agricultural landscape.

In the nineteenth century, however, the town was extensively farmed. Dairying was a major industry, with butter production a specialty. Maple syrup was produced in quantity in the spring. Lumbering brought extra income from the hardwoods beech, birch, and maple, and from the softwoods hemlock and spruce.[1] The south part of town had a copperas works. Cuttingsville, also in the south part, was the major shipping center because it was on the Rutland and Burlington Railroad, completed in 1849, and North Shrewsbury was the chief industrial center.[2]

Cold River and Mill River run through Shrewsbury from southeast to northwest, but both streams are near their sources, and only a few sites required spans long enough to have housed timber trusses of any type. Cuttingsville briefly had a covered bridge over Mill River, which is said to have been built by Timothy K. Horton, to whom the existing Kingsley Bridge near East Clarendon is also attributed. The same river downstream had a non-roofed but boxed-in wooden pony truss on a site long abandoned near today's Long Trail crossing. Finally, Brown Bridge still crosses Cold River in the heavily forested northwest part of town on Upper Cold River Road.

THE BUILDING OF BROWN BRIDGE

The early history of the Brown Bridge site is unknown, but the area saw its first major wave of settlement in the 1780s, and there seems to have been a crossing of Cold River at or very near the present site.[3] A ford may have been used at first; later bridges were probably of the pile-and-trestle or simple truss type, and not covered.

Shrewbury's nineteenth-century population peaked at 1,289 in 1830, then dropped slightly over the next few decades as families left for the West. It peaked again at 1,235 in 1880, then dropped sharply for many years.[4] The construction of Brown Bridge in 1880 coincided with the end of prosperity, but times were still good; the bridge is well built, and sports an expensive slate roof.

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[1] Hemlock, maple, and yellow birch predominate at the Brown Bridge site today.

[2] Hamilton, Child, *Gazetter and Business Directory of Rutland County, Vt., for 1881-82* (Syracuse, New York: Hamilton Child, 1881). See also Curtis B. Johnson, ed., *The Historic Architecture of Rutland County* (Montpelier: Vermont Division for Historic Preservation, 1988), p. 393.

[3] Dawn D. Hance, *Shrewsbury, Vermont: Our Town As It Was* (Rutland: Academy Books, 1980), p. 8, has a transcription of a 1788 survey map that is useful.

[4] *Inventory of the Town, Village, and City Archives of Vermont* (Montpelier: Vermont Historical Records Survey, 1940) XXII, no. 11, Rutland County, Town of Shrewsbury, p. 12.

Nichols M. Powers of nearby Clarendon, one of Vermont's best-known covered bridge builders, put up the present Brown Bridge very early in 1880. Preparatory work might have begun in late 1879.[5] The winter may seem an unusual time for construction, but many builders, including Powers, were also farmers, so winter was when they had time for bridgework.[6]

In the nineteenth century stonework was very expensive and often amounted to half or more of a bridge's cost. This was not the case at Brown Bridge. Powers carefully sited the crossing at a huge erratic boulder, which serves as the entire northwest abutment with the addition of a very small amount of dry-laid stone on top. Town records note expenses as follows:

Geo. Streeter, laying stone	7.00
R. Lloyd, laying stone	4.50
F.M. Plumley, labor on stone work	20.75
A. Knight, labor	6.00
Chas. Gleason, labor	4.37
N.M. Powers, 20 days' work	72.45
D.M. White & Co., lumber and nails	455.61
H.W. Wilcox for cash expended for labor and boarding help	398.30
G.W. Chaplin, for slate and laying same	109.39

The total cost therefore was $1,078.37, assuming that all of the stonework expense is included here. Some incidental expenditures are recorded for the previous year, but they were probably for last-minute repairs to the old bridge.[7]

Powers' fee of $72.45 may seem low, but an experienced builder could indeed have a covered bridge up in just three weeks.[8] It is interesting that the slate roof accounted for some 10 percent of the bridge's cost, more than Powers' fee. There was a tradition of using slate for bridge roofs in this part of Vermont as well as for the roofs of

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[5] The 1880 Shrewsbury Town Report, which covers the time period generally February 15, 1879 to February 15, 1880, documents construction. It is therefore possible that the bridge was built in 1879, but the long-accepted 1880 date probably came from the builder's grandson Gratz, and I am inclined to trust it. Powers required only three weeks on the job, and this could well have been in January. My only question is whether the housing including slate roof could have been applied fast enough for the costs to have been included in the 1880 report. It appears likely that the stonework, at least, may have been done late in 1879.

[6] At some sites it was also easier to build construction falsework on the ice of frozen rivers rather than in the flowing current at other times of the year, but this was probably not a factor here. Also, timber cut in the winter, when much of the sap is in the roots, is less subject to warping as it dries out. Many builders, or course, used seasoned lumber, but it appears that this was not always the case.

[7] *Annual Report of the Board of Officers for the Town of Shrewsbury* (commonly called Town Report), 1880. Though this is not specified, it appears that H.W. Wilcox was town agent for the project. The Town Meeting minutes were examined but offered no clues.

[8] James F. Tasker of Cornish, New Hampshire was another builder who could work quickly. See Richard Sanders Allen, *Rare Old Covered Bridges of Windsor County, Vermont* (Brattleboro, Vermont: The Book Cellar, 1962), pp. 32-33.

houses and even of barns. Just to the west, the Poultney, Vermont and Granville, New York area was a major slate quarrying center.[9]

The Shrewsbury town officers had good reason to select Nichols Powers as builder. They probably knew him personally, since he owned land in town.[10] For a modest investment he gave them a bridge, which is still serving traffic some century and a quarter later with very little modification.

STRUCTURAL DETAILS

The northwest abutment, as described above, makes use of a huge boulder. The southeast abutment is dry-laid random coursed stone. The bridge was slightly raised in recent years, and a new course of mortared fieldstone replaced some original bed timbers, but otherwise the old abutments are intact.[11]

The wooden trusswork is of the Town lattice plan, patented in 1820 and 1835 by New Haven architect Ithiel Town. Powers was familiar with other plans such as the Long truss and the Howe truss, but he usually favored the Town lattice. In Brown Bridge the lattice planks are nominal 3" x 10" with some manufacturing variations; they measure net 2-7/8" x 9-1/2" to 9-3/4". The upper chords, both primary and secondary, also use nominal 3" x 10" plank. The lower primary chord measures net 3" x 11-1/2", and the lower secondary chord is 3" x 11". This increased size shows sophisticated understanding on Powers' part and is intended to compensate for two problems. First, lower chords are in tension, and when wood is used in tension there is inefficiency at the plank joints. There are doubled chord sticks on both sides of the lattice web for a total of four thicknesses of plank. Where a plank joint occurs, the other three planks carry the entire load through that area. Powers' larger chord sticks make up for most of this loss of section. Second, the lower chords also bear the weight of the floor system and are subjected to bending moments in addition to tension. The upper chords require no such size adjustment because they are in compression, where the plank joints cause no loss of function, and because they do not carry the weight of the floor.[12]

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[9] The slate industry brought a large number of Welsh quarrymen to that area, which is today a major center of Welsh-American culture. Other notable covered bridges with slate roofs included the Billings Bridge of Rutland Town, which crossed Otter Creek on nearly the exact site as the U.S. Route 4 freeway today; and the former Dean Bridge of Brandon, which also crossed Otter Creek, on what used to be known as Clay Street but today is called as Union Street. If this seems extravagant, note that several Vermont towns once had marble sidewalks, since the quarries were nearby.

[10] Shrewsbury records for 1879 tell of a fence dispute involving Nichols M. Powers' land.

[11] The state engineers are to be commended for their trust in the old stonework. In earlier years there were regrettable cases in which such stonework was entombed behind concrete. The east end of the Hutchins Bridge in Montgomery once rested on a high outcrop of natural ledge; there, all is now hidden behind concrete.

[12] Field notes, June 3, 2002.

The treenails (wooden pegs) are 2” net, with two per lattice joint, four per chord joint. The truss length at the floor measures 112’-3”. The sides are tightly weatherboarded except for a small space at the top. Boarding also extends inside for a short distance at each end to protect against wind-driven rain. This arrangement is known as a shelter panel.

Floor beams are 5-1/4” x 11-1/4” net, placed through every lattice diamond, resting on both halves of the lower primary chord. Because of the close spacing, there is no need for stringers, and the deck of vertical plank is directly atop the floor beams. This system is not original. A 1972 inspection revealed floor beams of approximately 3” x 14”, many of them doubled. They were placed not only at every lattice diamond, but even at the midway points, where they could rest only on the inside half of the chord. Such a plan can produce racking in the lattice, and it too was probably not original. Ordinary floor plank rested on top.[13] There is no known record of the original floor system, but a good guess is 3” x 14” floor beams through every lattice diamond, resting on both halves of the chord, with no stringers, and with the plank floor directly on top.

The beautiful slate roof installed in 1880 is still mostly intact, although there is some evidence of patching since a few roofer boards have been replaced.

Brown Bridge has never carried heavy traffic, and it has required only normal maintenance. The deep-woods location posed a special challenge around 1970 when some animal, probably a porcupine, ate one of the lattice joints.[14] In 2002, Wright Construction Company replaced a few lattice planks and re-sided the bridge. It is in good condition and should long remain as a testimony to the skill of its famous builder.

THE TOWN LATTICE TRUSS

Ithiel Town of New Haven, Connecticut (1784-1844) is best known as an architect who popularized the Greek Revival style. He designed state capitols, churches, and other prominent buildings, some of which are still in existence. He was also a major figure in the history of bridge engineering, for he developed the first completely new idea in truss design since the Middle Ages, the Town lattice truss.

Before Ithiel Town, long-span bridges were built either as arches, panel trusses, or some combination of both. All required large timbers and much custom joinery. The Town lattice truss used standard sawn plank in a repetitive pattern, which could be built to any length, and made continuous over piers for added strength. It did not require complicated woodworking. There were no mortises, and it was held together by large

[13] The size of the former floor beams was obtained by estimating from an old photograph and comparing that with the size of the lower primary chord timbers, which was known from measurement.
[14] Field notes, September 18, 1973.

wooden pegs called treenails, pronounced "trunnels" and sometimes spelled that way. Town also saw the possibility of using bolts at the joints.

Town was working in North Carolina when he received his first bridge patent in 1820. The plan called for a single lattice, with simple chords at top and bottom. Some sources say that Town specified an angle of 45 degrees between lattice planks and chords, but in fact he said "about 45 degrees or any angle that may be necessary for a brace (as they do the office of a brace)." He designed the bridge to be covered, although he said it could be built instead of iron.

Experience soon showed that the original Town lattice plan, though strong, was subject to warping. Town added secondary chords to correct this problem. He described them in 1820s literature and included them in a revised patent in 1835 (No. X3169) covering a doubled lattice.[15] Although Town's papers were lost in the Patent Office fire of 1836, he was still actively promoting his plan, and was able to reconstruct the record.

Town built two covered bridges in North Carolina in 1818 and in 1819 that may have been prototypes for his lattice truss, plus one in his native Connecticut.[16] Apart from this he was a promoter of his "lattice mode" rather than a builder, deriving substantial income from his patent royalties. He also used a variant of his lattice for roof trusses in the First Presbyterian Church in Fayetteville, North Carolina, which still exists, and perhaps in other structures as well.

The Town lattice truss became a dominant style in covered bridges in the two areas where the inventor himself was active: New England and the South. Later builders brought it elsewhere and established regional traditions in other scattered areas. In modified form it was built up to the mid-1950s in Quebec, where the Department of Colonization still built covered bridges in new agricultural areas.

The Town lattice truss was one of the most widely used forms of timber trusses, and it was the favorite plan of Nichols M. Powers, builder of the Brown Bridge.

NICHOLS M. POWERS

Vermont's best-known bridge builder was Nichols Montgomery Powers of Clarendon. Historians know him as Nicholas Powers, and some of his contemporaries thought that was his name too, but it is clear that the great builder's first name was really

[15] Sometimes cited as 3169X. The X apparently denotes a patent reconstructed after the Patent Office fire. If you try to call up patent 3169, without the X, you will find something about wagon wheels that has nothing to do with Ithiel Town.

[16] On Town, see Richard Sanders Allen, *Covered Bridges of the Northeast* (Brattleboro, Vermont: Stephen Greene Press, 1957), pp. 15-16, and by the same author, *Covered Bridges of the South* (Brattleboro, Vermont: Stephen Greene Press, 1970), pp. 3-5.

Nichols.[17] Born on August 30, 1817 in a section of Pittsford that was later set off as part of Proctor, he lived for a time in Ira, but spent most of his life in Clarendon.[18] His name does not appear in any of the various biographical volumes of prominent business leaders published in the nineteenth century, either because of modesty or because he was not interested in paying for a subscription to be included. Fortunately, details of his life were preserved by his grandson Gratz Powers, who continued to live in the historic homestead in Clarendon into the 1950s and granted interviews to various historians.[19]

Powers' first bridge was a Town lattice truss over Furnace Brook at Pittsford Mills, Vermont, which he built in 1837 while still legally a minor. His father Richard Powers had to sign the business contract and promise to make good any spoiled timbers. There were no spoiled timbers, however, and the bridge lasted until 1931, safely carrying a 20-ton steamroller during construction of its replacement.[20]

Throughout the 1840s, Powers was busy building bridges in the Rutland area, sometimes by himself and sometimes in partnership with another builder. In 1855 he was called away to North Blenheim, New York to build what is today his best-known structure. Blenheim Bridge over Schoharie Creek, which is the longest single-span covered bridge in North America at 210' clear and with a total structure length of 228' (see HAER No. NY-331). It uses a modified Long truss, without the counterbrace wedges, but with the addition of a three-leaf timber arch through the central (counterbrace) plane of the center truss. It is a two-lane bridge, and this center truss reaches up to the ridgepole. Scoffers said that such a lengthy bridge would fall of its own weight when the construction falsework was removed. When the day came, Powers climbed to the roof and said that if the bridge went down, he would go with it. People

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[17] Richard Sanders Allen, in a letter to me some twenty years ago, first pointed out that the builder's name on his gravestone is Nichols M. Powers, not Nicholas. Child's *Gazetteer*, an 1881 business directory cited in these notes, lists him as Nichols M. Powers. Town records of Shrewsbury, where he owned land, refer to him the same way. His will is on file at the Rutland Probate Court under Nicholas M. Powers, but it is clearly signed in the builder's own hand as Nichols M. Powers. His wife, as executor of his estate, also refers to him as Nichols M. Powers. Curiously, his own grandson Gratz Powers referred to him as Nicholas.

[18] One source says August 3, but the gravestone says August 30. C. Ernest Walker published directions to the Powers birth site in *Covered Bridge Topics*, July 1958, p. 4, which he said he got by "accurate checking with an interested descendant." They are as follows. "If one takes the Upper Florence road from Proctor, he passes Beaver Pond on the upper left and continues to the top of a hill. Here he will find a narrow pasture on his right and a spur track of a railroad close by on his left. In the middle of the pasture there is a deserted cellar-hole, half filled with debris, with a small tree growing in one corner. This spot…is Nicholas Powers' Birthplace." I visited the site early in 1974 and found it still as described. A repeat visit on June 3, 2002 revealed surprisingly little change. The railroad track is now a snowmobile trail, and the pasture is overgrown with blackberries and raspberries and has several white pines up to 24" diameter or more. The place is still recognizable, but there are no historical markers. The land is private, but not posted.

[19] Gratz' recollections are subject to the limitations of oral history, but they are invaluable. The were published in various newspaper accounts of Powers' career, of which collections may be consulted at the Vermont History Center in Barre, or the Rutland Historical Society in Rutland. See Richard Sanders Allen's excellent account of Powers in *Covered Bridges of the Northeast*, pp. 50-54.

[20] Pittsford Mills is at the southern end of Pittsford village. The bridge site is on U.S. Route 7 just north of the junction with State Route 3.

then said that the bridge would sag so much as to be useless. Powers replied that if this happened he would jump off. When the falsework was taken away the bridge settled only slightly, even less than Powers had calculated.

In 1866 Powers traveled to Maryland to work as a boss carpenter on a huge railroad bridge over the mouth of the Susquehanna River between Perryville and Havre de Grace, Maryland; Amtrak crosses at approximately the same site today. A tornado destroyed the nearly completed bridge, and the construction superintendent was fired. While the new superintendents dallied over the plans, Powers was asked to produce something. After just a few hours he had drawn plans, carpenter-like, on a large block of wood, and received the job. He called his sixteen-year-old son Charles from Clarendon as assistant, and had him do most of the complicated work of laying out the draw span. Finishing ahead of schedule he collected a generous bonus, but did not stay for the opening ceremony because his wife Lorette wanted him back home to run the family farm.[21] His letter home from Havre de Grace still exists, and draw a remarkable portrait of the man. He obviously enjoyed the job, and was very proud of his teenaged son Charles, who especially relished the work. In his quaint phonetic spelling he told his wife, "If you could sea this work A going and the place I hold I think you would tell me to stay till the job was done. I am treated with more respect in wone day than I would in Clarendon in on year but when the great draw is done I can cum home if you think best Charles hates to go home dreadfully…."[22] Charles went on to become a bridge builder himself, and was responsible for at least two bridges in Maine, but he died young at the age of thirty.[23]

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[21] I do not wish to detract from the great builder's stature, but old accounts imply that he was the engineer/architect of Havre de Grace Bridge, while it is clear from contemporary records that he was really construction superintendent. The first bridge, which blew down, was a Howe truss with arches, and there is a rough pencil sketch of it by Powers himself among the Powers papers at the Vermont History Center in Barre. The second bridge, of which Powers was superintendent, was of the exact same design, and from Powers' own correspondence we know that he used much of the timber from the destroyed bridge to build the new. George A. Parker was probably the engineer/architect. The distinction between engineer, architect, and builder was less clear for smaller bridges, just as today houses are sometimes both designed and built by carpenters, with no assistance from an architect.

[22] Powers papers, file MS-62 at the Vermont Historical Society Library, Vermont History Center, Barre. His wife's name appears to be Lorette in correspondence, but it is Loriett on her gravestone in Ira, Vermont. Her maiden name was Fish, and the Powers grave is next to that of Preserved Fish, Esquire. This, however, is not the same Preserved Fish as the famed New York City steamship owner and politician.

[23] Charles Powers' career as a builder deserves to be better known. The Powers papers contain a letter from Charles to his father written from a job site in Old Town, Maine. Unfortunately the year is missing. He mentions that the job had been delayed because a shipment of angle blocks had not arrived, so the bridge was probably a Howe truss. His boss was a Mr. Collins. Around 1868, railroad bridges were under construction across Penobscot River, from Old Town, across Treat and Webster Island, to Milford, Maine; this may be the job meant. Another letter, which appears to be from Nichols Powers, also mentions Mr. Collins, and was written from Buxton (Maine); the year is missing. One old source says that Nichols Powers had contracted to build the famed Dorr Bridge of Rutland, but became ill and Charles finished the job in 1872; however there is not general agreement on this fact, some other sources list Evelyn Pierpont as the builder.

Afterwards, Nichols Powers did not leave Vermont again, but he continued as a bridge builder until 1880. The Town lattice truss was his plan of choice, and he was fond of saying that a stick of his favorite spruce timber was stronger than an equivalent weight of iron. From his papers, though, it appears that he was thinking of iron design late in life, and there is even a plan for a dam involving concrete. He worked in other areas of industrial design, especially railroading, although little is known of this aspect of his career. He also built marble derricks for the quarry industry, and probably mills and other projects. Though not formally educated, Powers had a natural head for mathematics and could do complicated calculations without writing them down.[24] In an era when college-trained engineers were taking over the building profession, Powers represented an older craftsman tradition, and he more than held his own.

In addition to his building activities, Powers worked a large farm at his home in Clarendon, and also had a cheese factory.[25] He died on January 17, 1897, and is buried at Ira, where his stone clearly gives his name as Nichols M. Powers. His five remaining covered bridges, including Brown Bridge, are an even more elegant monument.

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[24] Recollections of great-grandson Russell Fish Powers, interviewed by Joseph D. Conwill in May 1974 at Rutland, Vermont.
[25] Hamilton Child, *Gazetteer* (see note 2), p. 318. Those curious to learn more of Preserved Fish may consult p. 146.

APPENDIX A: LIST OF KNOWN BRIDGES BUILT BY NICHOLS M. POWERS

Those still in existence are marked with an asterisk *. Adapted from information kindly provided by Richard Sanders Allen.

1837	Pittsford Mills, Vt., Furnace Brook
1840	Mead Bridge, Proctor or Pittsford, Vt., Otter Creek with D.C. Powers and Abraham Owen
*1842	Gorham Bridge, north of Proctor, Vt., Otter Creek with Abraham Owen
1845	Parker Bridge, Clarendon, Vt., Cold River with Moses Chaplin
1845	Lester Bridge, north of Rutland, Vt., East Creek
*1849	Cooley Bridge, south of Pittsford, Vt., Furnace Brook
1849	one Twin Bridge, north of Rutland, Vt., East Creek
*1850	other Twin Bridge, north of Rutland, Vt., East Creek
1851	Railroad Bridge, Bellows Falls, Vt., Connecticut River (worked on with others)
1852	North Clarendon, Vt., Cold River, with Timothy K. Horton
1854	Schoharie, N.Y., Schoharie Creek (repair)
*1855	North Blenheim, N.Y., Schoharie Creek
1866	Railroad bridge, Havre de Grace, Md., Susquehanna River (worked on with others)
1869	Powers Bridge, Clarendon, Vt., Mill River
1874	Wallingford, Vt., Otter Creek
1876	76' Bridge, north of Rutland, Vt., East Creek
*1880	Brown Bridge, Shrewsbury, Vt., Cold River

Also, he built a number of covered wooden railroad bridges on the Bennington & Rutland Railroad. Powers may have had a hand in building the Mill Village Bridge north of

Rutland over East Creek, an the 1875 bridge over Poultney River at Poultney, Vt. There is some uncertainty as to which of two Mead Bridges he built in 1840. The 1850 Rutland Twin Bridge exists on dry land in use as a shed.

APPENDIX B: ENGINEERING REPORT

ABSTRACT: The objective of this study was to gain a structural understanding of the Town lattice-truss, specifically as found in the Brown Bridge. The scope of the study involved first-order linear elastic analysis of the truss, but did not include analysis of specific connections. Research revealed that the truss closely followed general beam behavior, having chord forces that corresponded to the bending moment distribution in a beam and diagonal forces that corresponded to the shear distribution in a beam. Maximum stresses were found to occur at the diagonals and lower chord in the immediate area of the first interior support. This indicated that a bolster beam is critical to the longevity of such bridges. Also considered were issues of structural efficiency versus constructional efficiency.

AUTHORS: Dylan Lamar, HAER Engineering Technician, Summer 2002, and Benjamin Schafer, Ph.D., Assistant Professor of Civil Engineering, Johns Hopkins University.

INTRODUCTION

This report focuses on the significant engineering aspects of the Brown Bridge. The bridge's historical context, in terms of engineering technology, will be briefly explored, as well as a discussion of design and construction methods of the period. The main portion is a structural analysis of the Town lattice-truss form as found in the Brown Bridge to gain a more definitive structural understanding of this bridge and others like it. Additionally, the structural efficiency of the truss system and the advantages of its configuration will be considered.

HISTORICAL CONTEXT

As J. G. James notes in his article, "The Evolution of Wooden Bridge Trusses to 1850," Ithiel Town, a famous Connecticut architect, first patented his lattice-truss in 1820. The first known bridge to utilize Town's truss form was built in 1823. The truss consisted of plank timbers in an interlocking lattice form with joints usually assembled using two treenails (wooden dowels, pronounced "trunnels") each. Continuous top and bottom chords were then added on each side of the lattice, also fastened with treenails. With regard to the behavior of these early Town trusses, James notes, "it is generally agreed that Town's early lattices were very prone to warp and some were given auxiliary bracing." [26]

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[26] J. G. James, "The Evolution of Wooden Bridge Trusses to 1850," *Journal of the Institute of Wood Science* 9 (December 1982): 172-175.

Perhaps due to this general lack of stiffness, Town modified the design in another patent in 1835. This patent described doubling the lattice so that the joints of one lattice would be out of phase with the other and the addition of a secondary set of top and bottom chords. While James mentions that the second patent "became standard for railway use when spans over about 120 ft were needed," most roadway bridges using the Town lattice-truss contain only a single lattice structure, but do include the secondary chords.[27] Joseph C. Nelson, in his book on Vermont's covered bridges, notes that in Vermont, "the bridges built with the four pairs of chords have held up well over the years," however, "only three of [those] built without upper secondary chords survive … all three have required additional bracing."[28]

The lattice-truss became quite popular as it was "aggressively promoted" by Town. It is known that "Town made his fortune not by building bridges himself, but by selling the rights to use his design."[29] The popularity of his design stems from considerations of economy and construction. The Town lattice-truss is significant because it can be built quickly by unskilled labor and without using large-dimension timber.[30] In contrast, the Burr arch-truss (one of the competing bridge forms of the day) calls for rather tedious methods of timber joinery and very large timbers, typically no smaller than 6" in any dimension. Large timbers were more expensive than the smaller planks of Town's truss, usually having a smaller dimension of 3". Additionally, all joints of the lattice-truss were made with treenails, which do not require the skillful carpentry work of traditional timber joinery or expensive metal bolts. However, this is not to say that the treenail joints required less effort. Nelson reports that in one 100' bridge, over "2,500 holes must be drilled to receive nearly a thousand treenails."[31] The Brown Bridge required over 3,000 holes.

The Town lattice-truss does not represent any significant advancement in terms of structural understanding or efficiency, but is rather an incremental change. The design is structurally redundant to a large degree, which makes the basic design robust but inefficient by today's standards. Compared to a Burr arch-truss, a type also studied in this project, a Town lattice of the same span weighs about 10 percent more. While this may not seem like a significant difference, it can have a considerable economic and structural effect. The economic effect is obvious, as more weight equates to more consumption of costly raw material. The structural effect centers primarily on the creep response of timber structures under sustained loads. As timber bridge specialist Jan Lewandowski notes, dead load tends to have more critical effects on a timber bridge than its live load, due to the constant presence of dead load.[32] The effects of loads on timber vary greatly with time, as is represented in today's National Design Specification

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[27] James, 175.

[28] Joseph C. Nelson, *Spanning Time: Vermont's Covered Bridges* (Shelburne, Vermont: New England Press, 1997), 249-50.

[29] Nelson, 248-9.

[30] Nelson, 250.

[31] Nelson, 249.

[32] Jan Lewandowski, interview by author, July 2002.

(NDS).[33] For example, the NDS prescribes reducing a member's capacity by 10 percent for dead load. Therefore, the extra dead weight of the Town lattice-truss can have a substantial effect.

However, aside from this unnecessarily increased dead load, the additional members of Town's truss create a significant degree of structural redundancy, which can be considered beneficial. As one timber framer is quoted:

> It's obvious that [the Town lattice-truss] is a good truss to use wood in, because wood is not predictable. Any one piece can be different by quite a bit! If you have hundreds of junctions like the plank lattice does, it doesn't matter if some pieces are weaker than others. In the queen-post truss, on the other hand, it matters a lot.[34]

A typical queen-post truss is shown in Figure 1. This design is not redundant because each member is critical to the stability of the structure. If one member fails, the whole structure fails. Thus, while it is highly efficient structurally, it is completely dependent on every one of its members for stability. Structural redundancy has both positive and negative effects to an engineer weighing design efficiency against safety.

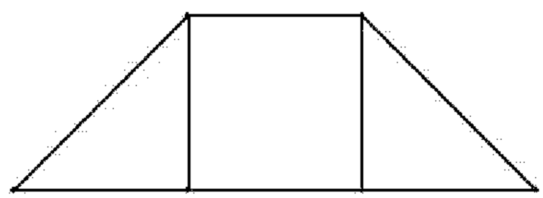

Figure 1. Queen-Post Truss.

The lattice-truss patented by Ithiel Town in 1820, along with its modifications of 1835, achieved considerable popularity among the early timber bridge builders. This is due predominantly to its ease of construction and economy, which often made it a more favorable choice than the Burr arch-truss patented just a few years previous. The continued use of Town's truss through the nineteenth century speaks for the truss' reliability—as James mentions, "following Town's death in 1844 … the relative simplicity and cheapness of his system ensured that such bridges continued to be built for several decades more."[35]

DESIGN AND CONSTRUCTION IN 1880

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[33] American Forest and Paper Association, American Wood Council, *National Design Specification for Wood Construction* (1997), 9.
[34] Nelson, 250.
[35] James, 176.

Successful historic engineering structures arouse a sense of awe for the skillful manner in which they were built in what we now often think of as a more primitive time. Nichols Powers certainly relied on his vast experience in Town lattice-truss bridge building to guide his design of the Brown Bridge, but bridge design was starting to change into a largely analytical process.

Design based on scientific engineering calculations steadily grew in popularity during the nineteenth century. Claude-Louis Navier developed one of the earliest methods of analyzing truss forms in 1826.[36] The method was based on the analogy of treating a truss as a simple, pin-supported beam. Navier's procedure reliably estimated the stresses in the chords of trusses and began to be used in the United States in the 1830s. Later, with Squire Whipple and Herman Haupt's publications on truss analysis, in 1847 and 1851 respectively, more advanced methods of analysis became possible.[37] However, Whipple and Haupt's methods were only competent for relatively simple, "statically determinant" structures, in which the internal forces in members depend only on the geometric location of the members, and not on each member's stiffness. Town lattice-trusses contain such a multitude of members with fixed joints that they are said to be "statically indeterminate." The forces in the lattice system depend on both the geometry and stiffness of the connected members. James Clerk Maxwell developed the first accurate methods of analyzing forces in such complicated, indeterminate structures in 1864, but it is improbable that Powers bothered with such technical procedures.[38]

Powers built his first bridge, a Town lattice-truss, in 1837, when he was just nineteen years of age. Forty-three years later, when the Brown Bridge was erected, he had considerable experience in timber construction. While he may have been aware of Maxwell's equations by then, the arduous task of analyzing forces and sizing members using them would have taken a long time and likely would have been no more useful than what he knew to be safe from experience. To summarize the comments of a trained engineer in 1895, when a skillful carpenter works with a certain truss over a course of years, he gradually refines the sizing of the members to the precise size suggested by engineering calculation.[39] He reasoned that timber as a material shows obvious signs of distress when it was overloaded, whereas cast iron, for instance, gave little evidence of distress until it ruptured. From this type of empirical evidence, Powers would have known which members were in tension and which were in compression, as well as the members that were most critical.

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[36] D.A. Gasparini and Caterina Provost, "Early Nineteenth Century Developments in Truss Design in Britain, France and the United States," *Construction History—Journal of the Construction History Society* 5 (1989): 22.

[37] Stephen P. Timoshenko, *History of Strength of Materials* (New York: Dover, 1953), 185.

[38] Russell C. Hibbeler, *Structural Analysis* 4th ed. (Upper Saddle River, New Jersey: Prentice Hall, 1999), 353.

[39] Jonathan Parker Snow, "Wooden Bridge Construction on the Boston and Maine Railroad," *Journal of the Association of Engineering Societies* (July 1985).

There is geometric evidence in the Brown Bridge that, even if no calculations were performed, Powers did have considerable structural understanding of his bridge. For example, the chords of the truss have relative sizes that roughly corresponded to the magnitude of forces they carry. While the individual timbers all have a standard nominal width of 3", their depths vary. The primary bottom chord has the largest depth, a nominal 12". The secondary bottom chord follows at 11", and both of the top chords have a nominal depth of 10". Powers understood that the bottom chords carried larger forces than the top, and that the primary bottom chord carrying the largest loads. Modern analysis reveals that, while less than optimum, this was considerably more efficient than simply assigning uniform depths to all of the chords. Another important consideration in Powers' design was his use of a bolster beam between the lower chord and the abutments. As seen in Figure 2, the bolster beam is able to cantilever from the support to slightly decrease the clear-span length of the truss, and it distributes the truss support over several points, rather than concentrating the forces through a single point, thus decreasing shear stresses and engaging several diagonals instead of only one.

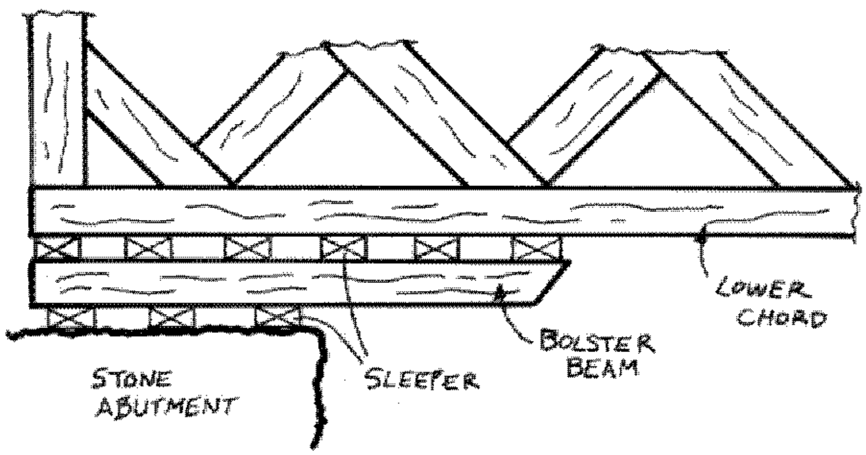

Figure 2. Typical Arrangement of Bolster Beam and Sleepers of Brown Bridge.

Another function of the bolster beam and sleepers, one perhaps just as important over the life of the bridge, is to provide replaceable members between the stone abutment and lower chord, as this area is prone to deterioration due to rainwater run-off.

There is no evidence that either Powers, or any other builder of a Town lattice-truss, ever attempted to optimally size or locate the lattice diagonals and improve structural efficiency, i.e., reduce the dead weight of the structure without decreasing structural capacity. In all known cases the diagonals are of uniform size and spacing. This possibility will be addressed later in this report, but it is safe to conclude that builders did not attempt anything but a uniform sizing or spacing of the lattice diagonals because it is the uniform nature of the truss that makes it efficient to construct. Indeed it has been said of the Town lattice-truss that:

> No attempt was made to use smaller sized material for members in areas of lower stress … any attempt to do so would likely have been lost in the complication of framing and erection that would have resulted from the use of varying sizes.[40]

The Brown Bridge is almost certainly based on empirical evidence from Nicholas Powers' many years of timber framing experience. His structural understanding is evident in his sizing of the chords and use of bolster beams, and his knowledge of construction techniques and efficiency is evident in his choice of form and use of uniformly sized diagonals with common joinery.

STRUCTURAL BEHAVIOR CALCULATIONS

The lattice-truss of Brown Bridge was modeled and analyzed using MASTAN2, a structural analysis computer program, assuming linear-elastic behavior.[41] The geometry of the bridge was input into the program based upon centerlines of the members, measured directly from the bridge in its current state (Figure 3). Section and material properties were then added to describe the individual members.

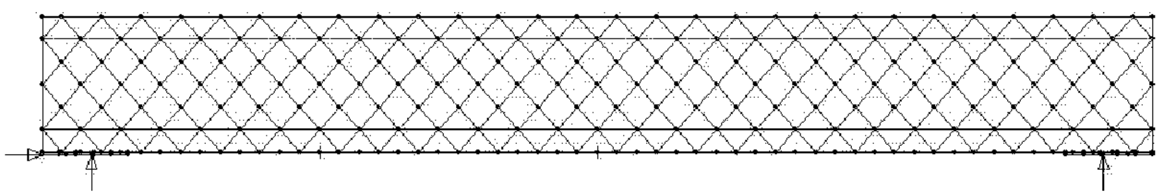

Figure 3. Center-line Two-dimensional Model of Brown Bridge.

Note the small vertical marks along the bottom chord in Figure 3, which represent the left quarter point and mid-point of the truss. These can be found in all the truss diagrams to easily identify these locations. Some of these diagrams contain only the left side of the span. This is a convenience that allows for larger illustrations and greater clarity when the behavior is symmetric about the center of the bridge.

Of particular note is the modeling of the bolster beam and supports. A photo showing the typical arrangement of the sleepers, bolster beam, and lower chord of the Brown Bridge is seen in Figure 4.

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[40] Donald O. Barth, "America's Covered Bridges," *Civil Engineering* (Feb 1980): 52.
[41] MASTAN2, version 1.0, developed by Ronald D. Ziemian and William McGuire, 2000

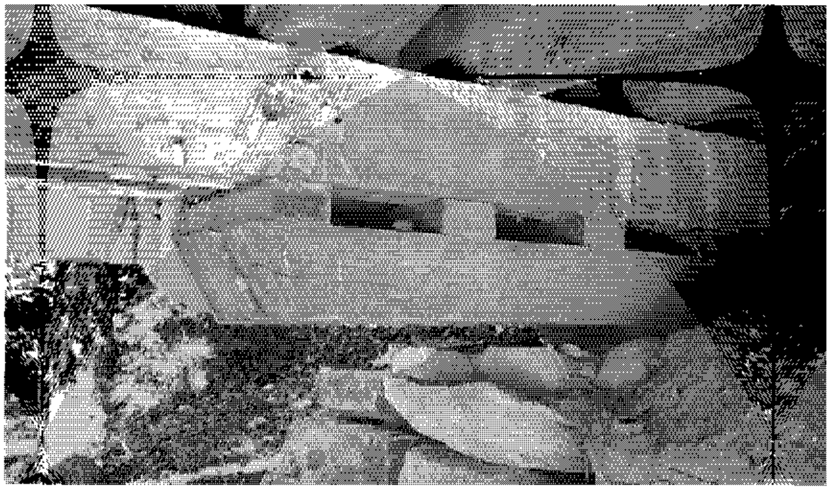

Figure 4. Bolster Beam between Bottom Chord and Stone Abutment of Brown Bridge.

Although all four original supports of the Brown Bridge were presumably identical, each one is currently unique. The bolster beam cantilever length ranges from 26" to 48", and the distance each beam is supported by the stone abutment also varies from 52" to 71". For the model, conservative averages of these values (45" cantilever length and 60" support length) were used symmetrically. A detail of the centerline model of the left support is shown in Figure 5.

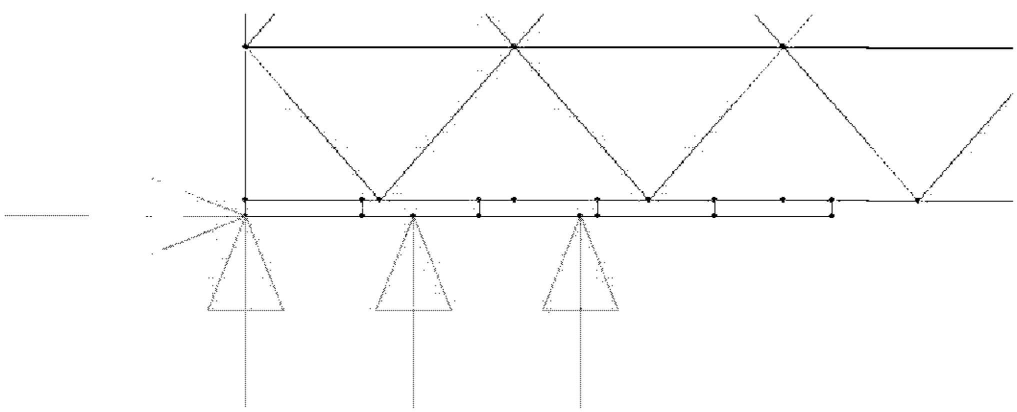

Figure 5. Detail of Centerline Model of Left-End Support (arrows indicate assumed support reaction points).

It soon became clear, however, that only the innermost vertical support acts in compression, with the outer two in tension. Since there is no possibility of a tension connection between the non-fastened sleepers, these supports were removed and the resulting model places only a single support at the innermost position. It should also be noted that the horizontal support reaction is placed only at the left end. Thus, the overall action is similar to that of a simply supported beam, having a pin at one end and a roller at the other.

Another minor difference between the actual structure and the model occurs at the ends. The model shown in Figure 3 places the end posts symmetrically at an intersection of the lattice diagonals. In the actual structure, this is not the case. The end posts are slightly farther in and not precisely at a lattice intersection. As a result, many of the treenail connections through the end posts to the lattice members are in such a bad shape as to be nonfunctional. It is postulated that this was not the original state of the bridge, but is the result of a later "patch job" of sorts, perhaps due to repair work to the approaching roadway or abutments that necessitated a shortening of the truss. Because of the low stresses in this area, the difference likely has little, if any, significant effect on the modeled behavior compared to the bridge's actual behavior.

To best approximate strength and stiffness characteristics of the timber, the wood species must be known. Based on a visual inspection of the bridge by experienced timber specialist Jan Lewandowski, it appears that Eastern Spruce was used for the main structural members.[42] Properties of this wood were obtained from the Forest Products Laboratory (FPL) and the National Design Specification (NDS).[43] The most important parameter for this model is the modulus of elasticity. While this value is highly variable, even among the same species, a value of 1,400 kilopounds per square inch (ksi) was selected for our model. A unit weight of 35 pounds per cubic foot (pcf) was assumed in calculating dead loads of the bridge. It was later found that the unit weight of Eastern Spruce at 12 percent moisture content is closer to 25 pcf.[44]

Maximum stress values for the suspected wood species of Eastern Spruce are seen in Table 1. As can be seen, there are two conflicting values given for each property. The NDS values are lower since these are "design values applicable to normal conditions of service," and account "for the effects of knots, slope of grain, splits, checks, size, duration of load, moisture content, and other influencing factors."[45] The values of the FPL, however, are based on an average of extensive high-quality-specimen tests, and do not include conservative adjustments.[46] While contemporary structures are required to have stresses below those designated as "maximum allowable" by the NDS, stresses in excess of these values are certainly possible up to the range prescribed by the FPL, and this is often observed in older structures.

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[42] Jan Lewandowski, interview by author, July 2002.

[43] Forest Products Laboratory (FPL), *Wood Handbook, Wood as an Engineering Material* (Madison, Wisconsin: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory, 1999), p. 4-12; American Forest and Paper Association, American Wood Council, *National Design Specification for Wood Construction (NDS)—Supplement* (1997), 39 (hereafter cited as *NDS—Supplement*).

[44] Perhaps the lower value would have been more appropriate, had it been known at the time of the analysis, but use of the higher unit weight resulted in a more conservative evaluation. Since the bridge's members can bear the stresses calculated using the higher value, substitution of the lower value would effectively raise the bridge's load rating or safety factor.

[45] *NDS—Supplement*, Introduction.

[46] FPL, p. 4-1.

Table 1. Maximum Strengths of Eastern Spruce.*

NDS Max Allowable Stress[47]			FPL Average Strength[48]	
Compression, // psi	Shear, // psi	Tension, // psi	Compression, // psi	Shear, // Psi
775	65	725	5560	1163

* "//" = Strength parallel to the wood grain (Shear strength parallel to grain is the limiting strength, even when loaded transversely.

A significant simplification in this model involves an idealization of the joints. The model allows for either of two cases to occur at the end of an element: perfectly rigid (fixed) or perfectly free to rotate (pinned). Since there are at least two treenails at each lattice intersection (See Figure 6), the rigid condition seems to be most applicable, and so was used in the model. However, this does not take into account the small non-elastic deformations that inevitably occur at the bearing surfaces of the treenail holes. These allow small rotations, which, on a large scale, can result in significant differences from a perfectly rigid modeling assumption. Since any rotations in joints like these are not perfectly free, but constrained in an unknown fashion, any attempt to model them would be exceedingly complex and of unknown validity. Nonetheless, it is believed that this model captures actual behavior closely enough for the general behavior of the bridge to be examined.

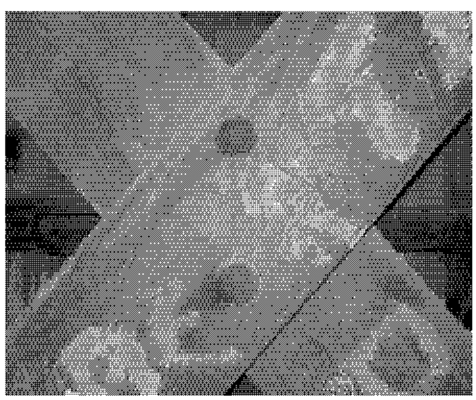

Figure 6. Photo of Typical Lattice Connection Using Treenails.

[47] Values shown are tabulated design values—they do not contain adjustment factors for safety or resistance and therefore are only approximate.

[48] These values are an average of the values given for Black, Red, and White Spruce, as "Eastern Spruce" is not listed. Also, FPL values of tension parallel to grain are available only for a select number of small specimens, which are not reliable for large timbers.

Measuring lumber dimensions on site, including truss members, roofing, siding, etc., provided approximations of dead loads.[49] The volumes calculated were multiplied by the unit weight of 35 pcf. The weight of the slate roof was also estimated from a current design code.[50] The loads were then placed at the nodes of the model in a manner that best approximated the actual loading condition. Live load was modeled to resemble the actual truck used in field tests. This dump truck, which the local agency of transportation provided, was found to weigh 19,940 pounds (lbf), so this was used for the live load in our model. This live load was equally divided among nine consecutive bottom chord nodes, as seen in Figure 7.

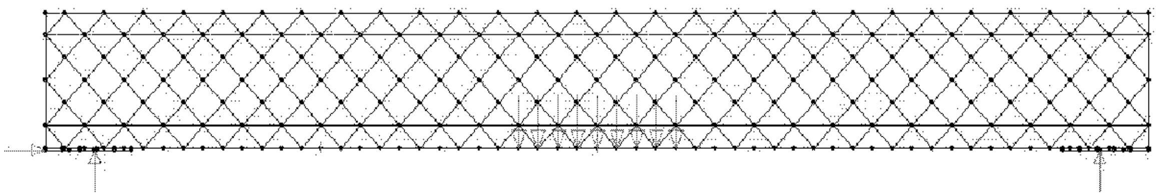

Figure 7. Live Load Distribution Over Nine Nodes (Mid-Span Loading Shown).

This live load distribution only approximated the truck's actual weight distribution, which was, of course, concentrated at its wheels, but the bridge's deck was not explicitly included in our model, and it was assumed that the deck was stiff enough to sufficiently distribute the concentrated wheel loads to the assumed uniform distribution with sufficient accuracy for the truss analysis.

Three live-loading conditions were investigated. The first was mid-span loading as seen in Figure 7. Field-measured values of deflection were compared with those calculated by the model. Similarly, the deflection due to quarter-point loading predicted by the model was compared to field measurements. Finally, end-span loading, with the live load located just inside the bolster beam cantilever (see Figure 8) were analyzed.

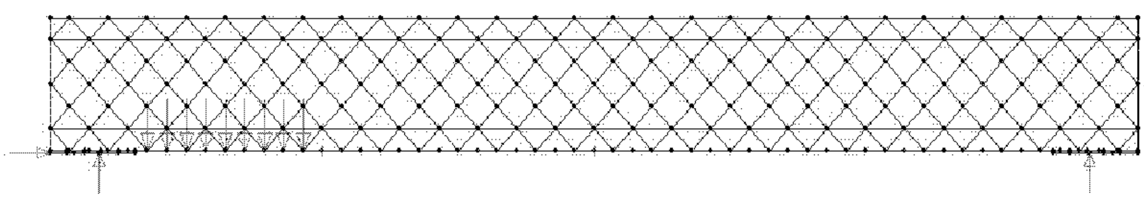

Figure 8. Location of End-Span Live Load.

Note that when referring to locations of the truss in the text of this report as well as in the data sheets, three general areas will be designated: the middle region (M), the quarter point area (QP), and the end region (E). These refer only to the general area and not a specific point. Due to the large number of elements, it would be overwhelming to

[50] American Society of Civil Engineers, *Minimum Design Loads for Buildings and Other Structures (ASCE 7-98)* (New York: ASCE, 1998), 230.

label each one. Rather, the type of element will be listed followed by the general location. The types of elements, as labeled in Figure 9, are: primary top and bottom chords, secondary top and bottom chords, inclined diagonal (those which are inclined upward toward mid-span), reclined diagonal (those which are reclined upward away from mid-span), end post, and bolster beam. The figures will aid in determining exactly which element is referred to.

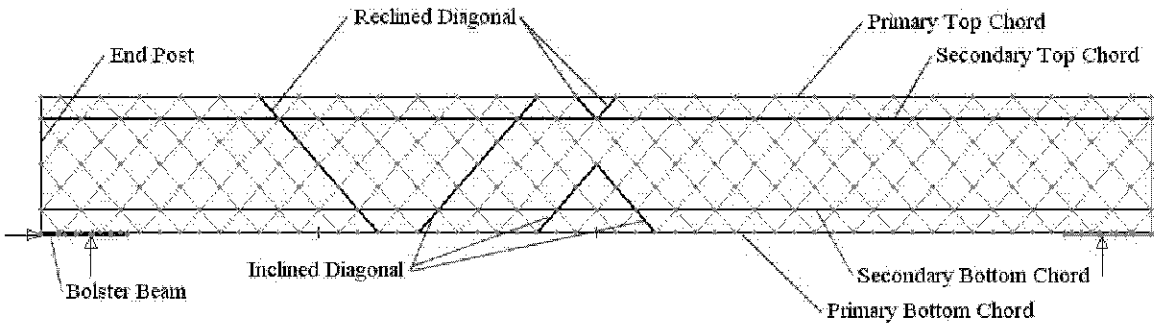

Figure 9. Truss Element Labeling System.

One additional characteristic of the lower chords must be noted to understand the stresses in the truss. In typical Town lattice-trusses there are no splice connections of the chords, where one timber ends and another begins. Rather, the ends are simply butted against one another; as can be seen in Figure 10. As noted in a recent article by Phillip Pierce, such a joint may transfer compressive forces, but it cannot transmit the tension forces expected in the lower chords.[51] Instead, the butt joints in the chords are staggered so that the remaining, continuous timbers may carry the load across the joint. Since each chord consists of four timbers, where one ends there remains only 75 percent of the total cross-sectional area to transmit the tensile force. Treenails transfer some of these forces between the chord members at adjacent joints with diagonals. As discussed in the article, this results in complex stress distributions among the individual chord timbers and diagonals. This complexity proved to be beyond the scope of this report, and the stresses reported in the chords considered their full area. However, it should be understood that these tensile stresses are greater in the continuous members around these joints.

[51] Phillip C. Pierce, "Those Intriguing Town Lattice Timber Trusses," *Practice Periodical on Structural Design and Construction* 3, no. 3 (August 2001): 92-94.

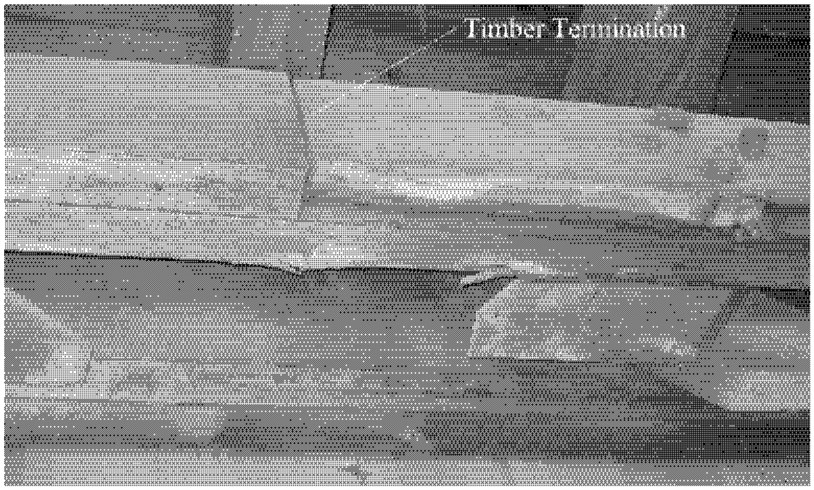

Figure 10. View of Primary Bottom Chord from Below Showing Timber Termination.

For reference, a summary of the main forces and stresses in the various truss members for each situation examined herein is provided at the end of this report.

FIELD MEASUREMENTS

During summer 2002, some field measurements were taken to supplement the computer model.[52] Deflection measurements, resulting from a 19,940-pound truck provided by the local agency of transportation, were taken. The truck was positioned at the mid-point and the quarter-point of the span in two runs, and the resulting deflection of the lower chord of the bridge was measured using surveying equipment.[53] The transit sighted prisms hung beneath the bridge as seen in Figure 11. Elevation angle and distance were measured with and without the live load of the truck, and the resulting deflections were calculated. Only when the data was examined after completion of the tests was it discovered that the prisms had swung back and forth in a transverse arc. The vertical deviation of the prisms resulting from this slight swing was enough to negate the reliability of these deflection measurements. The mid-span deflection was 0.20 ± 0.05 inch while the quarter-point deflection was 0.23 ± 0.05 inch. All known analytical methods will yield a mid-span deflection greater than a quarter-span deflection under these conditions, and the bridge showed no evidence of joint deterioration or movement that could possibly explain such unexpected behavior. Therefore, it must be concluded that the measurements were excessively inaccurate.

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[52] In 1994, the Vermont Agency of Transportation engaged the engineering firm of McFarland-Johnson of Binghamton, New York, to perform extensive field-testing of the Brown Bridge. See Vermont Agency of Transportation, McFarland-Johnson, Inc. "Covered Bridge Study at Brown Bridge." 1995. The Vermont Agency of Transportation furnished several pages of the McFarland-Johnson report, however, no formal conclusions from the testing were included, and the data proved too cryptic to yield useful conclusions for this report.

[53] Sokkia SET2-110 Electronic Total Station.

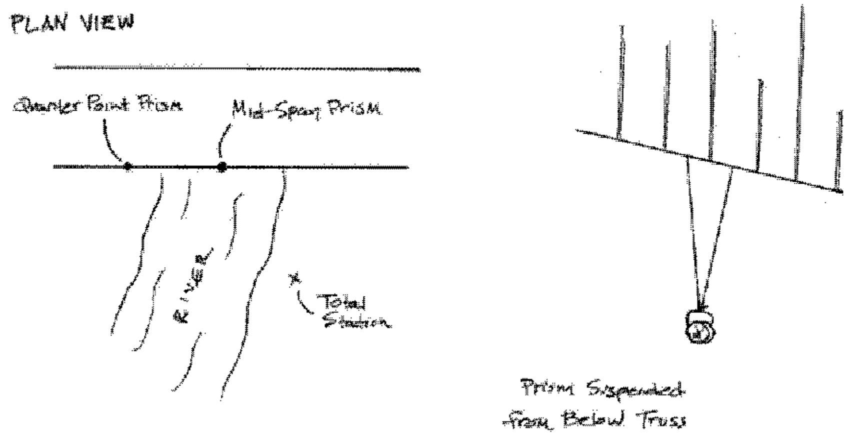

Figure 11. Arrangement of Field Measurement Apparatus.

DEAD LOAD BEHAVIOR

A common manner of conceptualizing the structural behavior of a truss is to think of an analogous beam. Indeed, one of the earliest means of approximating the chord forces in a statically indeterminate truss, developed by Navier in 1826, was based on just such an analogy. Through statics, one can calculate the shear forces and bending moments in a beam under various loadings. For example, Figure 12 displays the shear and moment diagrams for a beam placed under uniform dead load, represented by the series of arrows pointing down.

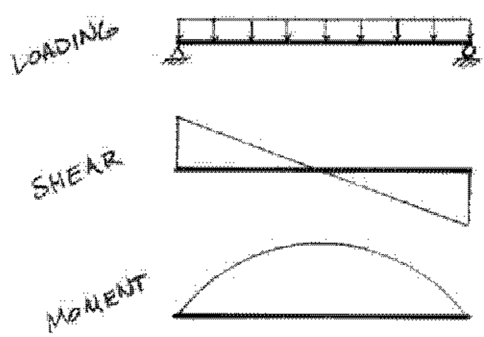

Figure 12. Shear and Bending Moment Diagrams for a Beam under Uniform Load.

The shear has a maximum magnitude at the ends and the moment is greatest at mid-span. The internal forces in a truss follow the "global" demands of Figure 12. Shown in Figure 13 is the axial force diagram of the Brown Bridge truss under dead load (a uniform load).[54] Shaded areas below or to the right of a member correspond to compression forces, and those above or to the left correspond to tensile forces. Thickness is an indication of the magnitude of the force in that portion of the member.

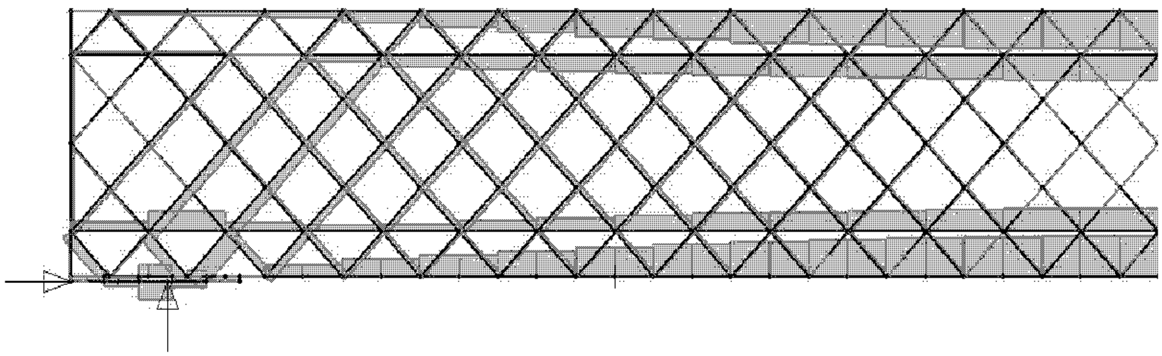

Figure 13. Axial Force Diagram due to Dead Load.[55]

The axial manifestations of the global shear and moment demands in the diagonal and chord forces are evident. Where the shear of a solid beam is greatest at the ends, so it is globally in the truss, as represented in the larger forces of the diagonals at the ends. Where the moment of the beam is greatest at mid-span, so it is in the truss, as represented in the larger forces in the chords at mid-span. Thus, the structural behavior of the truss can easily be conceptualized; the chords act as a force couple carrying the bending moment, and the diagonals transmit forces between the chords in order to keep them from shearing, or sliding past one another.

Due to this global beam behavior, uniform loading, such as dead load, causes the top chords to be in compression while the bottom chords are in tension. The inclined diagonals are in compression, the reclined diagonals are in tension, and the end posts see only small compressive forces.

Of course there are peculiarities to the truss that are incongruent with this beam analogy. Most significantly, there are stress concentrations at the supports. For example, in the secondary bottom chord there is a large force elicited toward the end. This occurs due to stress concentrations in the diagonals and also because the support conditions of the model involve the complications of the bolster beam and are not as simple as a single pin. The stress concentrations of the diagonals just above the first support are significant. The stress in the inclined diagonal is twice as large at this point as at any other section of it. Due to these stress concentrations, the greatest stress in the secondary bottom chord occurs here at the ends, rather than at mid-span.

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[54] It should be emphasized that this diagram represents only how the forces of the dead load are carried through the structure; it says nothing about stress. Those members carrying the greatest force are not necessarily under the greatest stress since the various members have different cross-sectional areas.

[55] Due to symmetry, only the left half of the span is shown.

The reclined diagonals also exhibit differences from the beam-analogy. Although normally in tension, there is a load reversal at the ends, which puts the reclined diagonals under significant compression. Just above the first support the reclining diagonal under compression carries twice the force of any tensile diagonal. Also, in the diagonals at the ends of the truss, outside the support, the forces quickly diminish and do not exhibit the high magnitude of the global shear experienced at the ends of solid beams.

It is interesting to see the amount of force the chords carry relative to one another at mid-span. The primary chords carry approximately equivalent forces, since they form a force couple resisting the global moment. At mid-span, the secondary top chord carries a force equal to 67 percent of that in the primary top chord. This is precisely what would be predicted by the elastic flexure formula, which predicts the stress induced in a solid beam due to bending (or flexure):

$$\sigma = \frac{M \cdot y}{I}$$

where σ = axial (longitudinal) stress, M = moment, y = vertical distance from the neutral axis (the centerline in this case) of the beam cross-section to the point of interest (chord centerline in this case), and I = moment of inertia, a property of the cross section's shape related to rigidity. Therefore, the stress at any point in the cross section of a beam is directly proportional to its distance from the centerline (neutral axis) of the beam.[56]

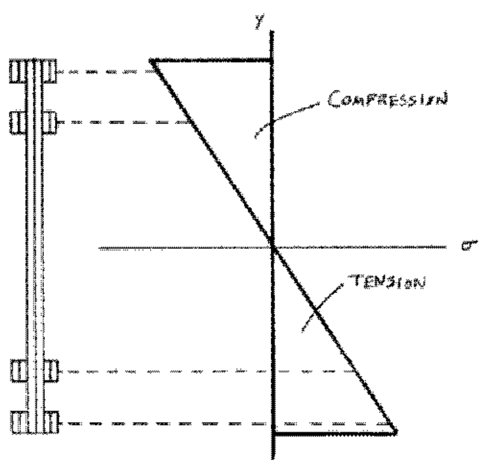

Figure 14. Moment Distribution in Truss Cross Section, Assuming Beam Behavior.

Figure 14 shows the stress distribution along a beam cross-section as predicted by the elastic flexure formula and compares it to the vertical profile of the truss. Since the secondary chords are exactly two-thirds (0.67) of the distance from the center to the

[56] Ferdinand P. Beer and E. Russell Johnston, Jr., *Mechanics of Materials*, 2nd ed. (New York: McGraw-Hill, 1992), 191.

primary chords, it would be expected that they carry a force directly proportional to that carried by the primary chords.

While this phenomenon does occur in the secondary top chord, other forces affect the secondary bottom chord. The weight of the floor, applied to the primary bottom chord, is transmitted through the diagonals to produce small amounts of compression in the secondary bottom chord (like global bending action, but locally between the pair of bottom chords). This compression negates some of the tensile force in the secondary bottom chord, and consequently that chord does not carry a full 67 percent of the load of the primary bottom chord, but only 57 percent of it.

Table 2 contains the maximum stress values as well as the maximum deflection calculated under dead load.[57] Considering the NDS limit of -775 psi for compression parallel to grain, a section of the reclined diagonal near the end, at -1329 psi, is substantially over-stressed by today's standards.

Table 2. Maximum Values due to Dead Load.

Max Compressive Stress, psi	-1329	Reclined Diagonal, End
Max Tensile Stress, psi	635	Reclined Diagonal, End
Max Deflection, in	-0.60	Mid-span

In Figure 15, the local shear forces of the truss elements are displayed. It is readily apparent that the only members containing significant shear are those near the support.

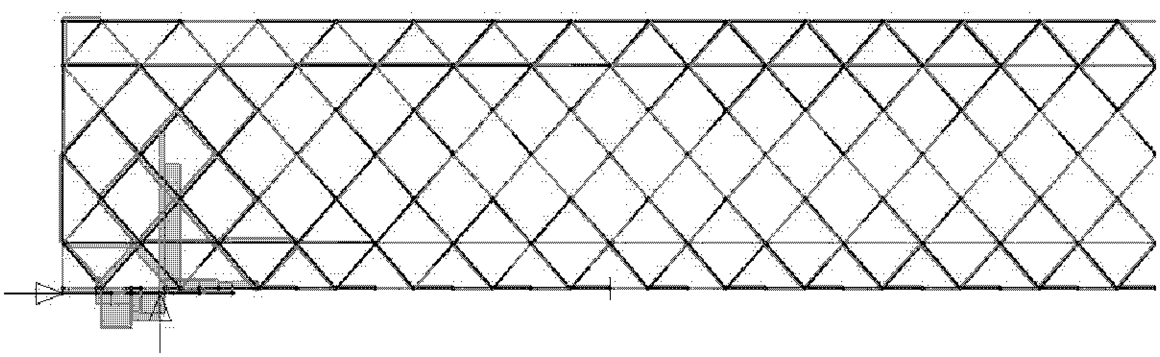

Figure 15. Shear Force Diagram due to Dead Load.

The greatest shear stress (248 psi) occurs in the bolster beam, followed by the primary bottom chord (189 psi). The NDS lists the shear limit as 65 psi, however, the average shear stress strength listed by the Forest Products Laboratory is 1,163 psi. Therefore, although they are not favorable, values in excess of the NDS limits are quite possible. Additionally, the NDS makes a specific exception: "shear design at supports for built-up components … such as between web and chord of a truss, shall be based on test or other

[57] All axial stresses documented in this report consider the extreme fiber of the member and include the effects of moment.

techniques [rather than based on the NDS limit]."[58] The NDS recognizes its own inability to predict allowable shear strengths near the supports of trusses, where compressive stress concentrations alter the timber's shear strength. Therefore, although the shear stress predicted by this model exceeds allowable NDS stresses, the NDS itself states that the allowable shear stress value is not applicable in the case considered. Recall also that the model assumes an essentially rigid bolster beam and, thus, considers all of the support force to act through this single point. In reality, the bolster beam distributes this force over several points, reducing this calculated maximum force considerably.

Figure 16 displays the local bending moments of the truss elements. Again, only the area around the supports sees significant values. The maximum moment occurs near the point of support in the primary bottom chord at a magnitude of 14,000 foot-pounds. The same moment causes a significant contribution to the stress in the bolster beam at the support.

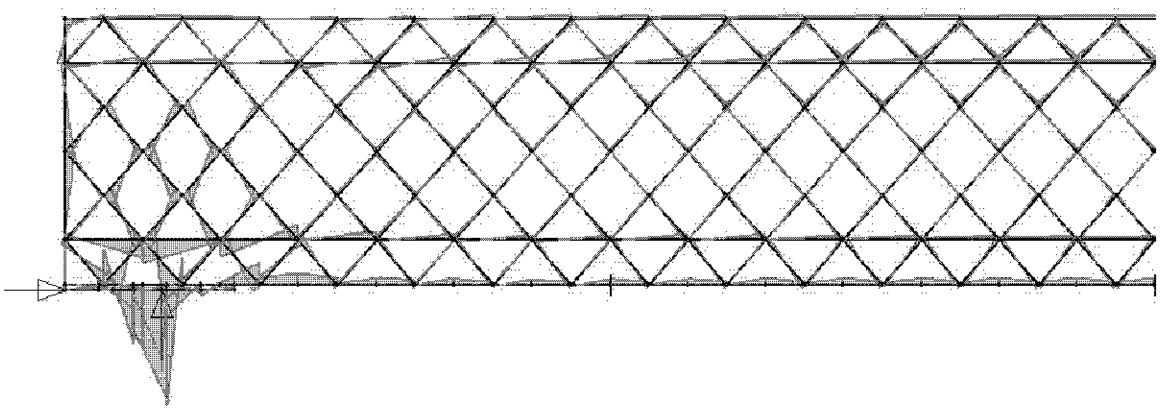

Figure 16. Bending Moment Diagram due to Dead Load.

MID-SPAN LIVE LOAD

Considering only the 19,940-pound live load at mid-span, and neglecting the effects of dead load, the global behavior of the truss is again found to be similar to a simple beam. The shear and moment diagrams of a simply supported beam under mid-span load are shown in Figure 17, followed by the calculated axial force diagram in Figure 18.

[58] *NDS—Supplement*, 17.

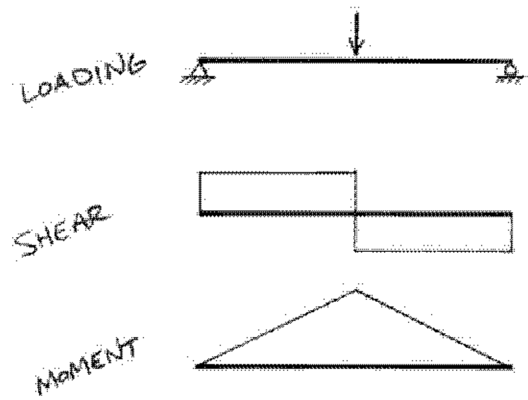

Figure 17. Shear and Bending Moment for a Beam Under Mid-Span Concentrated Load.

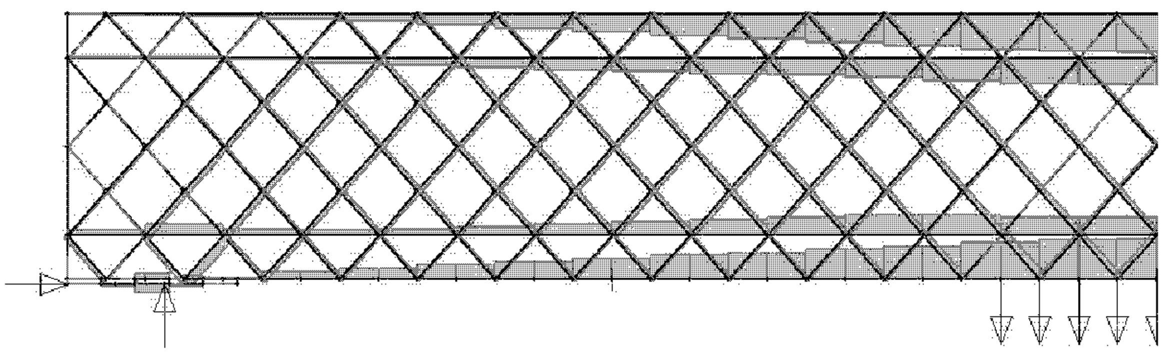

Figure 18. Axial Force Diagram due to Mid-Span Live Load.

As expected from the beam analogy, the chord forces follow the global moment, as they have a maximum magnitude at mid-span, with some variance evident at the support. The deviation from the beam-analogy behavior at the end of the secondary bottom chord is not as significant here as it was under dead loading. Under mid-span loading there is less global shear than dead load, which results in lesser end-diagonal forces. When the end diagonals receive less force, they consequently induce less force on the secondary bottom chord, compared to the forces produced in the dead load case. There is a deviation from the beam-analogy behavior evident in the secondary bottom chord at mid-span, as well, due to the compressive effects induced in the secondary bottom chord from the diagonals due to the mid-span live load.

The distribution of forces in the diagonals also follows the beam-analogy, as the magnitudes of these forces follow the global shear, with fairly uniform values of force throughout. Again, exceptions include significant force concentrations at the support, a load reversal in the end reclined diagonals, and force concentrations in the diagonals at the mid-span, near the points of live loading.

Table 3 contains the maximum stresses and calculated deflection for this loading condition. As shown, the maximum deflection of the model was -0.16 inch. This is less than the -0.20 ± 0.05-inch deflection observed in the field. Other than the errors noted in the physical measurement process, possible reasons for this difference are numerous, including assumptions of the material stiffness, and assumptions regarding joint rigidity. For instance, it is possible that the assumed value of 1,400 ksi for the modulus of elasticity is too high. With a value of 1,200 ksi it was found that the calculated mid-span deflection is -0.19 inch. This is certainly a more favorable result, but it is impossible to be sure that this is the reason for the discrepancy. All members were also assumed to have the same modulus of elasticity. Given the known variations in wood, it may be that consideration of a random distribution of material stiffness would have resulted in a slightly more flexible overall response. Another possible culprit is the simplification of modeling the timber connections as absolutely rigid. However, considering the scope of this project, the agreement is reasonable and the basic behavior of the truss is captured in this model. In particular, the behavior of the truss and the magnitudes of the stresses appear to be acceptably accurate for understanding the truss's structural behavior.

Table 3. Maximum Values due to Mid-Span Live Load.

Max Compressive Stress, psi	-195	Reclined Diagonal, End
Max Tensile Stress, psi	142	Primary Bottom Chord, Middle
Max Deflection, in	-0.16	Mid-span

DEAD LOAD PLUS MID-SPAN LIVE LOAD

Dead load behavior dominates the combination of dead and mid-span live load. The total dead load is about *seven times* the live load. Since the performed analysis is linear elastic, the reactions to any combined loading are simply a linear combination (sum) of the reactions to the individual loadings. For instance, the maximum compressive force in the primary top chord for the combined loading is exactly equal to the sum of the forces due to dead load alone and live load alone. The same is the case for deflection. Table 4 displays the maximum values for this loading.

Table 4. Maximum Values due to Dead Load Plus Mid-Span Live Load.

Max Compressive Stress, psi	-1524	Reclined Diagonal, End
Max Tensile Stress, psi	737	Reclined Diagonal, End
Max Deflection, in	-0.76	Mid-span

This loading results in the largest deflection of the analyzed load cases, but it is only -0.76", quite small for such a long timber span. By way of comparison, the Timber Construction Manual recommends a deflection limit of $L/300$ for highway bridge stringers, where L equals the span length.[59] In this case, the calculated deflection is only $L/1600$.

QUARTER-POINT LIVE LOAD

Although quarter-point loading does not produce any maximum stresses, it was a convenient location at which to apply the live load and measure the resulting deflection. The field measurement yielded a deflection of -0.23 ± 0.05 in. at the quarter point for quarter-point loading. As discussed above, this did not reasonably compare to the deflection of -0.09 in.—less than half of that which was measured—calculated by the model. The modeling of the bolster beam versus its actual geometry would have a greater affect on the quarter-point reading than the mid-span reading. This could account for the fact that the quarter-point discrepancy is greater than the mid-span discrepancy. Alternatively, we are left with the conclusion that the measured values are inaccurate, however, seeing no obvious flaw in the testing method, we have chosen to retain the measured deflection values in this report, despite its obvious inadequacies.

END-SPAN LIVE LOAD

The shear and bending moment diagrams for an equivalently loaded solid beam are shown in Figure 19, and the axial force diagram of the truss under end-span live loading, along with a detail, is shown in Figure 20 and Figure 21.

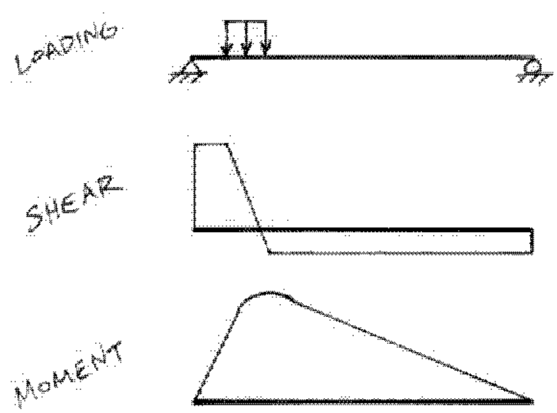

Figure 19. Shear and Bending Moment Diagrams of a Beam Under End-Span Distributed Load.

[59] Donald E. Breyer, Kenneth J. Fridley, Kelly E. Cobeen. *Design of Wood Structures, ASD*, 4th ed. (New York: McGraw-Hill, 1999), p. 2.21.

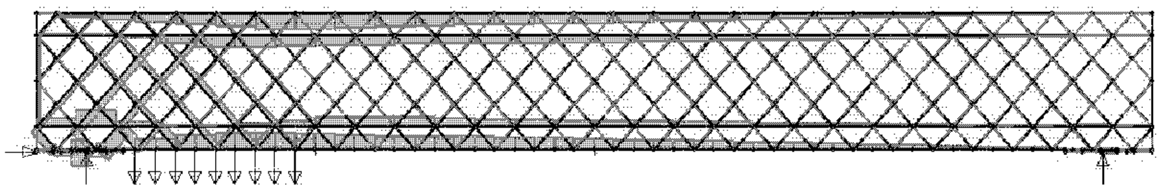

Figure 20. Axial Force Diagram due to End-Span Live Load.

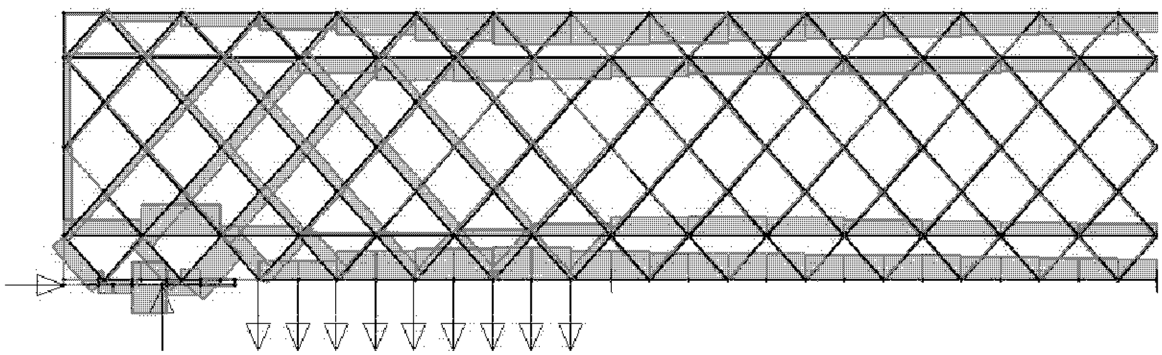

Figure 21. Detail of Left Side of Axial Force Diagram due to End-Span Live Load.

The force pattern in the chords mirrors the magnitude of the global moment, although, again, there is deviation from this analogy near the support. Also, in the secondary bottom chord the behavior deviates from the beam analogy considerably near the points of live loading. The global shear is largest to the left of the point of loading and uniform, but much smaller, to the right. The forces in the diagonals directly reflect this, but deviate near the support. Maximum stresses and calculated deflections for this loading are contained in

Table 5.

Table 5. Maximum Values due to End-Span Load.

Max Compressive Stress, psi	-339	Reclined Diagonal, End
Max Tensile Stress, psi	217	Reclined Diagonal, End
Max Deflection, in	-0.06	Mid-span

DEAD LOAD PLUS END-SPAN LIVE LOAD

For dead load plus end-span live load, the combined loading results are a linear combination of the previous results, with the dead-load reaction dominating. Comparing this load combination with the combination of dead and mid-span live load, it is seen, as the global shear and moment diagrams suggest, that diagonal stresses (global shear) are greater in the end-span loading case and chord stresses (global moment) are greater in the mid-span loading case. Consequently, this loading case produces the greatest compressive and tensile stresses of any studied. These occur in the reclined diagonals just above the support on the loaded end. It is interesting that these maximums occur in the diagonals rather than in the chords, suggesting that, for the selected member sizes, the critical members in this Town lattice-truss are the diagonals immediately above the first support, not the chords, as might be expected.

Table 6 contains the maximum stresses and calculated deflections for the combined dead and end-span live load. The overall maximum values in compression and tension shown are 136 percent and 29 percent greater, respectively, than the maximum allowable NDS stresses. This load case also produces the greatest shear stress of any condition studied; 338 psi in the bolster beam. Although these values represent significant over-stressing by today's standards, they are indeed possible considering the FPL values previously noted.

Table 6. Maximum Values due to Dead Load Plus End-Span Live Load.

Max Compressive Stress, psi	-1827	Reclined Diagonal, End
Max Tensile Stress, psi	933	Reclined Diagonal, End
Max Deflection, in	-0.72	Mid-span

STRUCTURAL EFFICIENCY

After analyzing the various loading conditions, the structural efficiency was examined by considering the maximum relative stress on each of the chords compared to the actual cross-sectional size of the members. The maximum relative stress was found by assuming all members to have the same cross-sectional area. In this manner, effects of moment were included, and the greatest stress in each type of chord under all of the loading conditions was found. Plotting these values against the equivalently proportioned actual member sizes yielded Figure 22. The values shown are percentages of the optimum (100 percent) stress and size of the primary bottom chord.

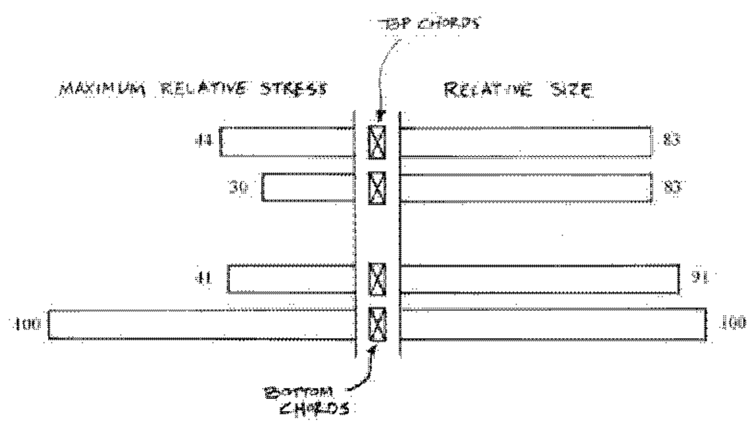

Figure 22. Efficiency of Chord-Member Sizing.

From this, the much-larger magnitude of maximum stress in the primary bottom chord, compared to the other chords, is readily apparent. A more-efficient size distribution (one which closely matched the relative stress distribution) would feature either an increased cross-sectional area of the primary bottom chord, or decreases in the area of the other three chords.

Typically, one can consider the truss behavior to follow the analogy of a beam, as shown for each of the loading cases. The beam analogy suggests that the upper and lower chords should be of the same cross-sectional size, as they undergo the same axial forces (from the global bending moment demands). The more detailed analysis completed in MASTAN, however, suggests that the primary bottom chord actually sees significantly greater forces than the top chords, due to stress concentrations at the supports. Interestingly, Powers' member sizing in his lattice-truss seems to reflect this fact, as the cross-sectional area of the bottom chord is slightly larger than the top chord. While our analysis suggests that even larger cross-sectional areas for the bottom chord would be more efficient, the fact remains that Powers appears to intentionally have used different member sizes for the chords. The chord member sizes Powers selected suggest a deeper, more-complex understanding of the behavior of a lattice-truss under loading than that available from the simple beam-analogy.

EFFECTS OF THE BOLSTER BEAM

The bolster beam is a common element in many wooden covered bridges. Indeed, during the 1988 restoration of the longest wooden covered bridge in the U.S., a Town lattice-truss, bolster beams having a 15' cantilever were installed.[60] By cantilevering from the abutment, a bolster beam helps to diffuse the large stress concentrations occurring near the support, particularly the large shear forces. Figure 23 shows a detail of the centerline model of the Brown Bridge, but without a bolster beam. The location of the supports is precisely the same as before, but they bear directly on the primary bottom chord.

[60] Teresa Austin, "Caring for a Covered Bridge," *Civil Engineering* (July 1991), 44, 45.

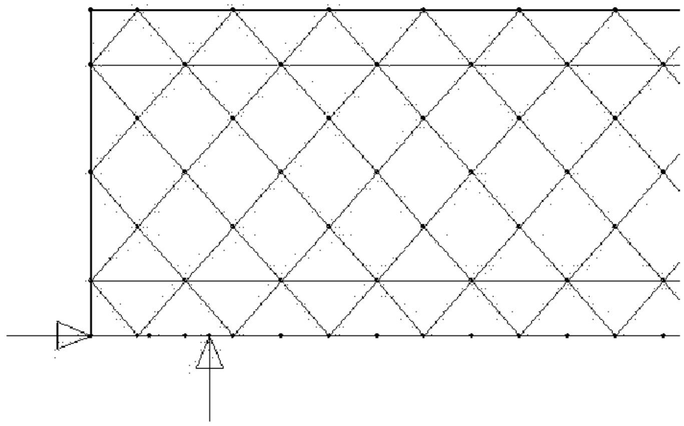

Figure 23. Detail of Centerline Model With No Bolster Beam.

A diagram of the axial forces in this system due to dead load plus mid-span live load is shown in Figure 24.

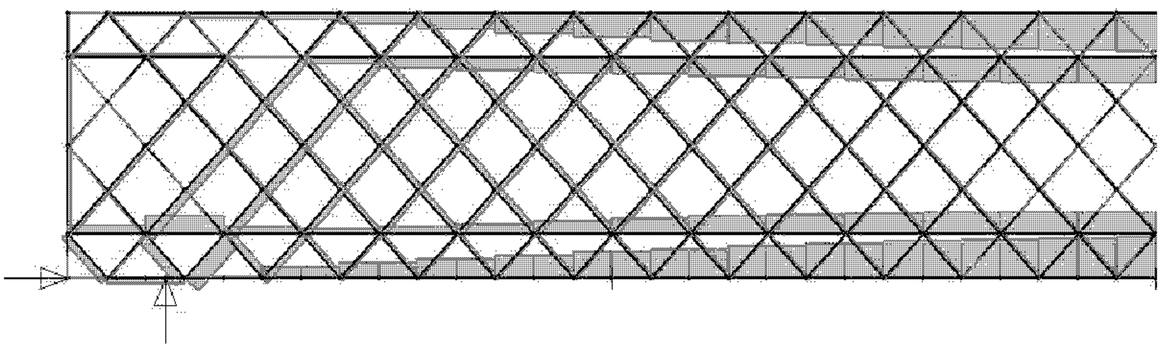

Figure 24. Axial Force Diagram of Truss Without Bolster Beam due to Dead Load Plus Mid-Span Live Load.

While there appears to be no significant difference in general nature of the axial force distribution, there are substantial changes in the magnitudes of many stresses, as seen in Table 7.

Table 7. Comparison of Maximum Stresses due to Dead Load Plus Mid-Span Live Load, With and Without the Bolster Beam.

Element [61]	With Bolster		Without Bolster		% Change
	Loca-tion	Axial Stress, psi	Loca-tion	Axial Stress, psi	
Primary Top Chord	M	-521	M	-521	0
Secondary Top Chord	M	-365	M	-365	0
Secondary Bottom Chord	M	291	M	291	0
"	E	388	E	427	10
Primary Bottom Chord	M	512	M	512	0
"	E	-849	E	-1295	53
Inclined Diagonals	E	-1017	E	-1296	27
"	E	-859	E	-1046	22
Reclined Diagonals	E	-1524	E	-1857	22
"	E	737	E	694	-6
Bolster Beam		-906	--	--	--

With removal of the bolster beam the chords and diagonals away from the support remain unaffected, as the load being carried is the same as before. However, near the end, large stress increases occur. Of particular concern are the maximum stresses in both the primary bottom chord and the diagonals near the end. Without the bolster in place these stresses increase markedly. In the worst individual case (the primary bottom chord) the increase is 53 percent. Further, the maximum stress in the entire model (a diagonal near the support) is increased 22 percent when the bolster is removed. Clearly, the bolster plays an important role in reducing the maximum stresses within members near the support.

EFFICIENCY OF THE SECONDARY CHORDS

As mentioned, Ithiel Town added a secondary row of chords to his truss in his second patent after many of the originals were "prone to warp."[62] Whether this refers to significant deflections, out-of-plane bowing, or both is not certain. What is interesting, however, is that these additional chord members were introduced in a secondary row, rather than simply added alongside the original primary chords. As discussed, if we assume the truss behaves as a beam, then we may use the elastic flexure formula which states that the axial stress in a beam is directly proportional to the distance away from its

[61] Stresses occurring due to the largest force in each element are listed initially. If effects of moment (or tensile force in the case of the reclined diagonal) result in greater or otherwise significant stresses they are listed and denoted with a ditto.

[62] James, 174.

neutral axis (which in this case is the center). Therefore, if one were to add material to the truss it would make sense to add it toward the outside, where the greatest axial stresses occur. The bending rigidity (I) of the truss also favors locating the additional members as far from the center as possible.

However, continuing with the beam-analogy, we find the shear distribution along a truss cross-section will yield a maximum at the mid-point, by the equation:

$$\tau = \frac{VQ}{It}$$

where τ = shear stress, V = shear force, Q = first moment of area (increases toward the mid-point of the cross-section), I = moment of inertia, a property of the cross section's shape related to rigidity and t = the thickness of the beam at the location at which you wish to know the shear stress. Then, for a beam of uniform thickness, the value of Q increases toward the mid-point of the web, and all other values are uniform. Therefore, the shear stress also increases toward the mid-point of the web, as seen in Figure 25.

The placement of the additional chord material closer to the center of the truss cross-section increases the thickness in this area, thus allowing for a greater resistance to the global shear stress demands in the truss cross-section. Further, the shear rigidity of the cross-section favors stiffening near the middle of the truss as well, where shear strains are highest.

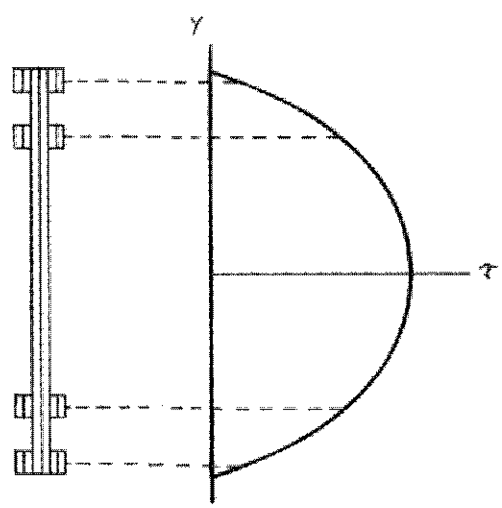

Figure 25. Shear Stress Distribution in Truss Cross-Section, Assuming Beam Behavior.

While demands of the moment distribution favor added material at the top and bottom of the cross-section, the shear stress distribution favors added material toward the mid-point of the cross-section. To examine these issues and discover which location is truly advantageous, consider a model of the truss where the secondary chords have been moved to the outside, i.e., added to the primary chords. We will refer to this as the "single-chord system." Loading the single-chord system model with the same dead and mid-span live load as the original truss configuration, we find that the maximum deflection is now -0.64"—less than the original truss deflection of -0.76". While this is certainly an improvement, we also find that the maximum stresses of the single-chord system are greater than those of the original truss by about 20 percent. The locations of greatest stress are in the diagonals, adjacent to the support. These diagonals receive significant axial loads and bending moments, as a result of the large global shear in this area. Therefore, the addition of the secondary chord does have significant positive effects in resisting the global shear of the truss. Adding the material to the primary chords, although providing a globally stiffer system, produces greater stresses.

However, there is another more practical side to this issue that must be considered involving efficiency of construction. If one were to add the material to the primary chord and replace the current 12"-deep bottom chord with a 24" member, this would seem to double the global moment capacity of the bridge. However, this is only true if the strength of the treenail connections could handle the full load of the 24" chords. To be sure, more treenails would be called for, and where they would be placed presents a problem, since the lattice diagonals already have four holes at their lowest intersection. Therefore, moving the additional material up to the next lattice intersection seems the easiest solution. In this way, the addition could be accomplished in the same manner as the primary bottom chord, using the same number of treenails.

Though it seems adding the additional chord material to the primary outside chords would increase truss stiffness, the stresses of the truss elements would be increased. To add the material closer to the cross-section's mid-point provides for a greater global shear resistance, resulting in lower maximum stresses, and also allows for more efficient construction.

EFFICIENCY OF THE LATTICE DIAGONALS

Another question of structural efficiency arises in reference to the lattice diagonals. In all loading cases which included dead load, the largest stresses in the diagonals occurred at the ends, due to the stress concentrations produced by the supports. Compared to the diagonal stresses near mid-span, the ends always contained a point of significantly higher stress. So, why do both locations contain the same amount of material? Certainly the answer is constructional simplicity, but it seems to be an inefficient use of material as well as adding an unnecessary contribution to dead load.

To explore this point further, consider an alternative design where the generally lower shear demands in the center are reflected in the structure by omitting every other diagonal near the middle region of the bridge, as shown in Figure 26.

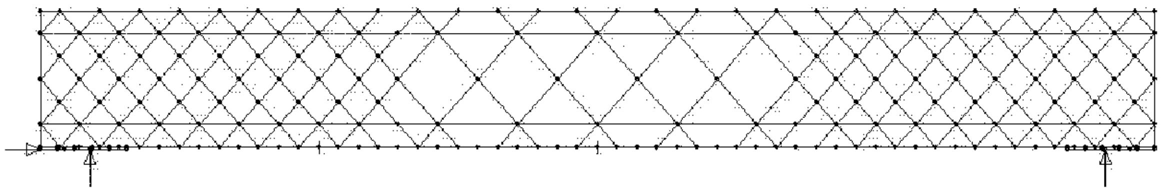

Figure 26. Model of Modified Lattice-truss.

The idea of the alternate design would be to achieve a decrease in dead load, presumably without a significant reduction in strength. An axial force diagram of the system under its approximated dead load and an identical mid-span live load as in Section 4.4 is shown in Figure 27.

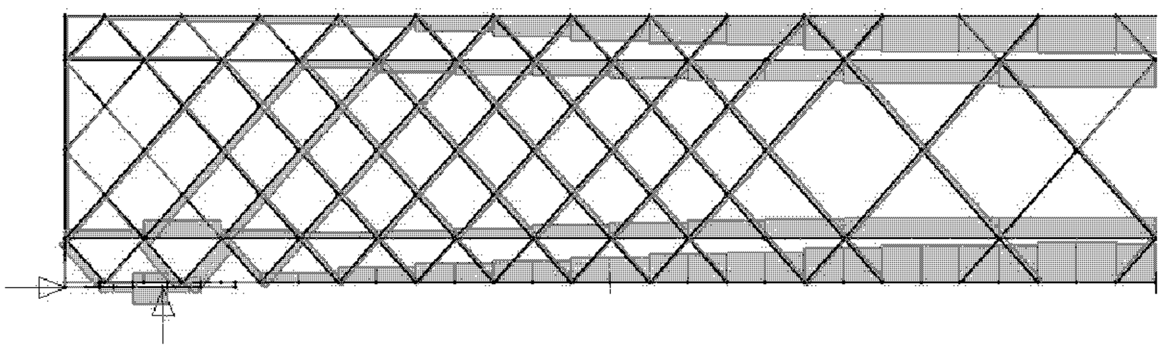

Figure 27. Axial Force Diagram of Modified Lattice due to Dead Load Plus Mid-Span Live Load.

Table 8 provides a comparison of the stresses of this system with the stresses of the original system under its dead load and the same mid-span live load. Larger forces begin to appear in the middle diagonals, but it appears that they are still no greater than those at the ends.

Table 8. Comparison of Maximum Stresses due to Dead Load Plus Mid-Span Live Load, For Original and Modified Lattice-trusses.

Element	Regular Lattice		Modified Lattice		% Change
	Loca-tion	Axial Stress psi	Loca-tion	Axial Stress psi	
Primary Top Chord	M	-521	M	-547	5
"	E	-522	M	-596	14
Secondary Top Chord	M	-365	M	-377	3
Secondary Bottom Chord	M	291	M	307	5
"	E	388	E	382	-2
Primary Bottom Chord	M	512	M	506	-1
"	E	-849	E	-837	-1
Inclined Diagonals	E	-1017	E	-1001	-2
Reclined Diagonals	E	-1524	E	-1502	-1
"	E	737	E	725	-2
Deflection	M	-0.76 in	M	-0.78 in	3

As Table 8 shows, the stresses in most cases do not significantly change. In the top chords, the modified system has larger stresses, and in others, most importantly the maximum stresses in the diagonals, the modified system has slightly smaller stresses. The deflection is also slightly increased in the modified system. It seems from the analysis, that this modified lattice has favorable results—less timber is used, fewer time-consuming treenail connections are required, and the maximum stresses of the system are not increased, and the deflection is only slightly greater. The top chord stresses are increased somewhat, although they are still within allowable limits. Of course, one would have to further examine this modified system through various load cases and by removing different diagonals to gain a better understanding of its value. However, the system is an interesting hypothetical modification reflecting ideas of structural efficiency and highlighting some of the issues brought out through our analysis.

CONCLUSION

The lattice-truss as patented by Ithiel Town in 1820 is a timber bridge design that nicely blends the two key issues of structural and constructional efficiency. Although wooden truss technology continued to expand, the Town lattice-truss was continuously utilized over a century after its invention. Indeed, Nichols Powers, the builder of the Brown Bridge, respected the design enough to use it throughout his career of over four decades.

Powers' extensive field experience as a builder determined the design and construction of the Brown Bridge. Evidence suggests his structural knowledge was derived from his experience as a builder rather than through scientific calculations. However, elements of the design of the Brown Bridge such as chord sizing and the bolster beam suggest that Powers' structural understanding was greater than a simple beam-analogy would have provided.

Structurally, the system acts much like a simple beam, having chord forces that correspond to the global bending moment demands, and diagonal forces that correspond to the global shear demands. However, significant stress concentrations occur near the supports, which, in the diagonals, result in stresses up to 136 percent greater than those allowed by the National Design Specification. Indeed, it was found that the critical members of the system are the diagonals immediately near the supports, not the chords. It was also found that the presence of the bolster beam greatly reduces these end-span stress concentrations.

Further examinations of structural efficiency versus constructional efficiency found that the addition of the secondary chords is more favorable than adding the same amount of additional material to the primary chords. Structural efficiency of the diagonals might be improved by the omission of selected diagonals from the middle region of the span, although further study is required in order to draw a fair and complete conclusion.

The Brown Bridge is an engineering landmark that recalls the time when engineering was not so far removed from construction. Instead, the two seemed to develop side-by-side. This is clearly evident in the case of the Town lattice-truss—a practical truss system that represents a harmonious blend of structural necessity and constructional efficiency.

SECTION & MATERIAL PROPERTIES

SECTION PROPERTIES [63]

Element	Width (x,y)	Depth (z)	Arrangement	Modeled Area[64]	Nominal Area	I_{zz}
Primary Top Chord	9.75	2.88	4-parallel	105.88	112.13	750.2
Secondary Top Chord	9.75	2.88	4-parallel	105.88	112.13	750.2
Secondary Bottom Chord	10.75	2.88	4-parallel	116.88	123.63	1012.92
Primary Bottom Chord	11.75	2.88	4-parallel	127.88	135.13	1330.77
Diagonal	9.75	2.88	single	26.47	28.03	187.55
End Post	9	2.88	4-parallel	97.63	103.50	586.18
Bolster Beam	8	8	2-parallel	120.13	128	601.25
Sleeper	6	17.25	single	97.75	103.50	269.32

MATERIAL PROPERTIES [65]

Modulus of Elasticity = 1,400 ksi Unit Weight = 35 pcf

[63] Areas and 2nd Moment of Area values (noted as I_{zz}) based on in-field measurements.
[64] Modeled area represents subtracting 1/8" from each face of wood to account for surface roughness.
[65] Modulus of Elasticity based on NDS and FPL values. Unit weight based on FPL data.

DEAD LOAD COMPUTATIONS

FOR EACH TRUSS:

AT PRIMARY TOP CHORD NODES [66]

	in^3 [67]	lbf/ft^3	lbf
Outer Top Chord	5393.21	35	109
Lattice Member	1022.17	35	21
Top Lateral Bracing	4355.70	35	88
Roof Structure	10967.08	35	222
Slate Roofing	6886.64 in^2	6 psf	287
Total			727

Extra at End Nodes from Overhang and Siding	350
Total	1,077

AT SECONDARY TOP CHORD NODES

	in^3	lbf/ft^3	lbf
Inner Top Chord	5393.21	35	109
Lattice Member	2044.33	35	41
Siding	2177.93	35	44
Total			195

AT LATTICE NODES

	in^3	lbf/ft^3	lbf
Lattice Member	2044.33	35	41

AT CENTER ROW OF LATTICE NODES

	in^3	lb/ft^3	lb
Lattice Member	2044.33	35	41
Siding	2549.95	35	52
Total			93

[66] All loads applied as concentrated loads.
[67] Volumes based upon nominal areas.

AT SECONDARY BOTTOM CHORD NODES

	in^3	lbf/ft^3	lbf
Inner Bottom Chord	5946.36	35	120
Lattice Member	2044.33	35	41
Siding	1238.58	35	25
Total			162

AT PRIMARY BOTTOM CHORD NODES (where lattice meets chord)

	in^3	lbf/ft^3	lbf
Outer Bottom Chord	3249.76	35	66
Lattice Member	1022.17	35	21
Siding	619.29	35	13
Bottom Lateral Bracing	7573.50	35	153
Flooring	15751.45	35	319
Total			572

AT PRIMARY BOTTOM CHORD NODES (between lattice-chord intersections)

	in^3	lbf/ft^3	lbf
Outer Bottom Chord	3249.76	35	66
Siding	619.29	35	13
Bottom Lateral Bracing	7573.50	35	153
Flooring	15751.45	35	319
Total			551

TOTAL BRIDGE DEAD LOAD PER TRUSS = 69,850 lbf

TOTAL BRIDGE DEAD LOAD = 139,700 lbf

FIELD-TESTING DATA

ORIGINAL POSITION

	Distance (ft)	Elevation Angle (deg)	
Mid-span	55.45	89.02611	
	55.47	89.02639	
	55.48	89.02639	
	55.46666667	89.02629667	
Quarter Point	*74.12*	*88.24139*	(*thrown out* [68])
	74.11	88.24694	
	74.13	88.24667	
	74.14	88.24694	
	74.12666667	88.24685	

LOADED POSITION

	Distance (ft)	Elevation Angle (deg)	
Mid-span	55.51	89.04389	
	55.48	89.04389	
	55.49	89.04389	
	55.49333333	89.04389	(Measured Second)

Deflection (in) = 0.20442932

55.49	89.04333	
55.49	89.04333	
55.49	89.04333	
55.49	89.04333	(Measured Third)

Deflection (in) = 0.19791634

AVERAGE EXPERIMENTAL MID-SPAN DEFLECTION (in)[69]:	0.20 ± 0.05
THEORETICAL MID-SPAN DEFLECTION (in):	0.16

Quarter Point	74.13	88.26056	
	74.13	88.26056	
	74.14	88.26056	
	74.13333333	88.26056	(Measured First)

[68] After this measurement was taken, the sight was aimed more precisely.

[69] It is assumed that the prism position was accurate to no more than 0.05” based on a 2” horizontal swing of the prism from a 36” radius.

Deflection (in) = 0.21285804

74.13	88.26333	
74.13	88.26333	
74.14	88.26333	
74.13333333	88.26333	(Measured Fourth)

Deflection (in) = 0.25586437

AVERAGE EXPERIMENTAL QUARTER POINT DEFLECTION:	0.23 ± 0.05
THEORETICAL QUARTER POINT DEFLECTION:	0.09

SUMMARY OF ANALYSIS DATA

Calculations based on:

Axial Stress, $\sigma = \frac{F}{A} \pm \frac{M \cdot y}{I}$ where F = axial force, A = cross-sectional area, M = moment, y = distance from neutral axis, and I = second moment of area.

Shear Stress, $\tau = \frac{V}{A}$ where V = shear force.

DEAD LOAD

Element [70]	Location [71]	Axial Force F, lbf	Shear Force V, lbf	Moment M, in-lbf	Axial Stress σ, psi	Shear Stress τ, psi
Primary Top Chord	M	-37456	0	-3567	-398	0
Secondary Top Chord	M	-24948	13	3851	-274	0
Secondary Bottom Chord	M	22535	15	5155	231	0
"	E	19890	592	30151	337	5
Primary Bottom Chord	M	39692	276	9681	371	2
"	E	-956	22783	169980	-742	189
Inclined Diagonals	E	-18765	134	-5136	-883	5
"	E	-1450	1170	27171	-746	47
Reclined Diagonals	E	-10244	1628	-36257	-1329	65
"	E	7551	630	-13132	635	25
End Post	Mid	-4004	274	13741	-146	3
"	Top	-1727	762	-10850	-100	8
Bolster Beam		-18489	29743	99129	-793	248
Deflection, Mid-span						-0.60 in

MID-SPAN LIVE LOAD

Element	Loc.	F, lbf	V, lbf	M, in-lbf	σ, psi	τ, psi
Primary Top Chord	M	-11552	0	-1152	-123	0
Secondary Top Chord	M	-8148	13	1442	-91	0
Secondary Bottom Chord	M	6005	57	1934	64	1
"	E	3055	94	4641	52	1
Primary Bottom Chord	M	12067	554	9710	142	5
"	E	-130	3304	24460	-107	27
Inclined Diagonals	E	-2894	18	-705	-134	1
"	E	-330	169	3947	-113	7
Reclined Diagonals	E	-1453	244	-5403	-195	10
"	M	2057	70	-1447	119	3
End Post	Mid	-592	44	2064	-22	0
"	Top	-87	111	-1594	-13	1
Bolster Beam		-2645	4257	14182	-113	35
Deflection, Mid-span						-0.16 in

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[70] Stresses occurring due to the largest force in each element are listed initially. If effects of moment (or shear) result in greater stresses they are listed and denoted with a ditto.

[71] M = middle region, QP = quarter point area, E = End region.

DEAD LOAD PLUS MID-SPAN LIVE LOAD

Element	Loc.	*F*, lbf	*V*, lbf	*M*, in-lbf	*σ*, psi	*τ*, psi
Primary Top Chord	M	-49009	0	-4719	-521	0
Secondary Top Chord	M	-33096	27	5293	-365	0
Secondary Bottom Chord	M	27732	22	7654	291	0
"	E	22945	686	34791	388	6
Primary Bottom Chord	M	51759	830	19390	512	7
"	E	-1086	26087	194440	-849	216
Inclined Diagonals	E	-21660	153	-5841	-1017	6
"	E	-1780	1339	31118	-859	54
Reclined Diagonals	E	-11697	1872	-41660	-1524	75
"	E	8697	736	-15324	737	30
End Post	Mid	-4597	317	15805	-168	3
"	Top	-1814	873	-12443	-113	9
Bolster Beam		-21134	34001	113310	-906	283
Deflection, Mid-span						-0.76 in

QUARTER-POINT LIVE LOAD

Deflection, Quarter-point	-0.09 in

END-SPAN LIVE LOAD

Element	Loc.	*F*, lbf	*V*, lbf	*M*, in-lbf	*σ*, psi	*τ*, psi
Primary Top Chord	QP	-5305	0	735	-58	0
Secondary Top Chord	QP	-3971	28	1223	-48	0
Secondary Bottom Chord	E	5323	177	8711	93	2
Primary Bottom Chord	QP	5387	598	10248	89	5
"	E	-202	5697	43730	-191	47
Inclined Diagonals	E	-4947	50	1605	-239	2
Reclined Diagonals	E	2981	173	3835	217	7
"	E	-2461	422	9471	-339	17
End Post	Mid	-1353	74	3530	-41	1
"	Top	-196	247	3534	-29	3
Bolster Beam		-4892	7401	25320	-204	62
Deflection, Mid-span						-0.06 in

DEAD LOAD PLUS END-SPAN LIVE LOAD

Element	Loc.	*F*, lbf	*V*, lbf	*M*, in-lbf	σ, psi	τ, psi
Primary Top Chord	M	-44653	5	-4252	-475	0
Secondary Top Chord	M	-29793	18	-4515	-327	0
Secondary Bottom Chord	E	27720	846	42635	472	8
Primary Bottom Chord	M	47266	277	10745	438	2
"	E	-1265	31181	233660	-1020	258
Inclined Diagonals	E	-26071	198	-7293	-1230	8
"	E	-2096	1609	37370	-1031	65
Reclined Diagonals	E	-13900	2250	-50139	-1827	90
"	E	11426	889	-18730	933	36
End Post	Mid	-5833	384	18953	-205	4
"	Top	-1992	1098	-15657	-139	12
Bolster Beam		-25535	40622	136020	-1089	338
Deflection, Mid-span						-0.72 in

TRUSS WITHOUT BOLSTER BEAM

DEAD LOAD PLUS MID-SPAN LIVE LOAD

Element	Loc.	F, lbf	V, lbf	M, in-lbf	σ, psi	τ, psi
Primary Top Chord	M	-49009	0	-4719	-521	0
Secondary Top Chord	M	-33096	27	5293	-365	0
Secondary Bottom Chord	M	27732	22	7654	291	0
"	E	23328	875	41506	427	8
Primary Bottom Chord	M	51759	830	19390	512	7
"	E	-7562	29785	285210	-1295	247
Inclined Diagonals	E	-23212	490	-14404	-1296	20
"	E	-1559	1615	38821	-1046	65
Reclined Diagonals	E	-13184	2302	-52448	-1857	92
"	E	9605	596	-12183	694	24
Deflection, Mid-span						-0.77 in

MODIFIED LATTICE

DEAD LOAD PLUS MID-SPAN LIVE LOAD

Element	Loc.	*F*, lbf	*V*, lbf	*M*, in-lbf	*σ*, psi	*τ*, psi
Primary Top Chord	M	-47223	0	-11550	-547	0
"	M	-45919	436	-21407	-596	4
Secondary Top Chord	M	-32864	51	7569	-377	1
Secondary Bottom Chord	M	26814	82	12215	307	1
"	E	22591	675	34259	382	6
Primary Bottom Chord	M	52027	368	-17331	506	3
"	E	-1072	25707	191630	-837	213
Inclined Diagonals	E	-21328	151	-5761	-1001	6
Reclined Diagonals	QP	7110	255	4956	411	10
"	E	-11529	1844	-41039	-1502	74
"	E	8566	724	-15072	725	29
Deflection, Mid-span						-0.78 in

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